

December 2009

Rt. 122 Highway Bridge Design and Construction

Arthur Hough Gager
Worcester Polytechnic Institute

Jason T. Carmichael
Worcester Polytechnic Institute

Follow this and additional works at: <https://digitalcommons.wpi.edu/mqp-all>

Repository Citation

Gager, A. H., & Carmichael, J. T. (2009). *Rt. 122 Highway Bridge Design and Construction*. Retrieved from <https://digitalcommons.wpi.edu/mqp-all/3484>

This Unrestricted is brought to you for free and open access by the Major Qualifying Projects at Digital WPI. It has been accepted for inclusion in Major Qualifying Projects (All Years) by an authorized administrator of Digital WPI. For more information, please contact digitalwpi@wpi.edu.

Rt. 122 Highway Bridge Design and Construction

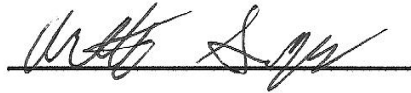
**A Major Qualifying Project Report
Submitted to the Faculty of
WORCESTER POLYTECHNIC INSTITUTE**

**In partial fulfillment of the requirements for the
Degree of Bachelor of Science**

By

A handwritten signature in black ink, appearing to read "Jason Carmichael", is written over a horizontal line.

Jason Carmichael

A handwritten signature in black ink, appearing to read "Arthur Gager", is written over a horizontal line.

Arthur Gager

Date: December 1, 2009

Approved by:

Professor Leonard Albano

Abstract

This project focused on the redesign of a typical two lane highway overpass. The basis of the design followed AASHTO specifications and used the LRFD method. To complement our hand calculations, we used Risa-2D to calculate live and dead loads. Along with designing the bridge, we performed a life-cycle cost analysis which included environmental impact, construction practices, and material selection.

Authorship

The abstract, authorship, background, methodology, recommendations, and most of the results were completed by both Arthur Gager and Jason Carmichael. We both put in equal amounts of time to complete the previously stated sections. The following design sections were done individually. These are:

4. Results

4.1.1. Composite Deck.....Jason Carmichael

4.1.2. Composite Girder.....Arthur Gager

4.1.3. Pre-Stressed Concrete.....Arthur Gager

4.1.4 Micropiles.....Jason Carmichael

Jason Carmichael



Arthur Gager



Capstone Design

Along with completing our Major Qualifying Project, we must complete our Capstone Design Requirement. This capstone design requirement followed the ABET guidelines and show our ability to design a system which included economic, environmental, social, health and safety, and sustainability aspects (Commission, 2007).

One goal of our project was to perform a cost analysis for both the initial construction and the life-cycle. This cost analysis showed the economical impact of this bridge. It also gave us an understanding on how different aspects of a construction project can affect the cost of a large scale project.

While we designed this bridge, we made sure to keep in mind the environmental impact of this project. This was done by lessening the initial impact the project has on resources. We also looked into using recycled building materials and recycling construction debris.

This bridge will have a large social aspect. Along with hopefully having less of an impact on the environment, this will affect the common people because it is necessary for traffic flow. Once this bridge is completed, it will improve the flow of traffic. Also, during construction, the affect the work has on passing motorists, bicyclists, and pedestrians must constantly be kept in mind (Highway, 2009).

In the design of the Rt. 122 bridge, we also considered public health and safety. This bridge followed AASHTO LRFD design, which is commonly used to design bridges. This system used load and resistance factors to ensure a stable bridge with sufficient strength. In addition to this, during construction of the bridge, we established practices for traffic control in

order to protect motorists, bicyclists, pedestrians, and construction workers on the site. Also, the current bridge has issues which make it unsafe to people who use it, such as low visibility and low clearance. We looked into ways at making these issues less prevalent.

Lastly, we focused on sustainability. Sustainability is the ability to meet the needs of the present without compromising the ability of future generations to meet their own needs (Commission, 2007). We looked at site design. One way to help with sustainability is through storm water management. We also looked at using local plants in the landscaping of the area once the bridge is complete. We also considered different ways to manage storm water runoff.

Contents

Abstract.....	1
Authorship	3
Capstone Design	4
Contents	6
List of Tables	10
1. Introduction.....	11
2. Background.....	12
2.1 Rt. 122 Highway Overpass.....	12
2.1.1 Issues with the bridge.....	13
2.1.2 Structural Problems.....	13
2.1.3 . Design Problems	14
2.2 Overview of Bridge Components.....	16
2.3 Wearing Surface	17
2.4 Bridge Deck.....	17
2.5 Primary Members	18
2.5.1 Rolled Beam.....	18
2.5.2 Pre-stressed Concrete Girders.....	19
2.5.3 Steel-Concrete Composite Box Girders.....	20
2.6 Secondary Members	22
2.7 Abutments	22
2.8 Piers.....	24
2.9 Bearings.....	25
3. Methodology	27
3.1 LRFD Design	27
3.1.1 Composite Deck Design	28
3.1.2 Composite Girder Design	30
3.1.3 Pre-Stressed Concrete	31
3.2 Cost Analysis.....	36
3.2.1 Initial Cost.....	36
3.2.2 Life -Cycle Cost.....	37
3.3 Construction Phase	38
3.3.1 Traffic Control	38
3.3.2 Site Design	40
4. Results.....	42
4.1 LRFD Results.....	42
4.1.1 Composite Deck.....	43
4.1.2 Composite Girder.....	47
4.1.3 Pre-stressed Concrete.....	49
4.1.4 Abutment Design	53
4.2 Cost Analysis.....	54

4.2.1	Initial Cost.....	55
4.2.2	Life-Cycle	60
4.2.3	Cost Summary.....	65
4.3	Construction Phase.....	66
4.3.1	Traffic Control	66
4.3.3	Material Selection	85
4.3.4	Site Design.....	86
5.	Conclusions and Recommendations	89
5.1	LRFD Design	89
5.2	Cost Analysis.....	90
5.3	Site Management.....	90
	Sources.....	92
	Appendix A: Proposal.....	95
	Table of Figures	97
	Abstract.....	98
1.	Introduction.....	99
2.	Scope and Methodology	100
2.1	Construction Phase	101
2.1.1	On-Site Preparation.....	103
2.1.2	Traffic Control	104
2.2	Material Selection	105
2.3	LRFD Design.....	108
2.4	Sustainability	111
2.5	Life-Cycle Cost.....	113
3.	Capstone Design	114
4.	Deliverables	116
5.	Conclusion	118
	Appendix B: Composite Deck Design.....	119
	Appendix C: Composite Girder Design.....	151
	Appendix D: Pre-stressed Design	155
	Appendix D: Bridge Inspection	163
	Appendix E: Life-Cycle Cost Break Down	173

List of Figures

Figure 1: Arial view of the bridge location (Google Earth).....	13
Figure 2: Center reinforced concrete pier (Google Maps).....	14
Figure 3: Damage From Collision	15
Figure 4: Superstructure and Substructure (Oklahoma Bridge Tracker, 2009).....	17
Figure 5: Rolled Steel I-Beams (Foam Laminates of Vermont, 2008).....	19
Figure 6: Pre-stressed Concrete Bridge in Denver, Colorado (Colorado DOT, 2009).....	20
Figure 7: Multi-cell Box girder System (New Crossing, 2008).....	21
Figure 8: Cross-Sectional Diagram.....	33
Figure 9: Loading.....	34
Figure 10: Dimensions of Rt. 122 Overpass.....	43
Figure 11: Composite deck illustration.....	47
Figure 12: Cross Section of Pre-stressed Bridge	49
Figure 13: Top View of Rt. 122 Overpass.....	58
Figure 14: Drainage Structures on Rt. 20 Eastbound Under Rt. 122 Overpass.....	63
Figure 15: Drainage Structures on Rt. 20 Westbound Under Rt. 122 Overpass	64
Figure 16: Detour for southbound traffic on Rt 122 (taken from Google Earth)	68
Figure 17: Areal view of Rt. 122 showing obstacles for a temporary bridge (Google Earth).....	70
Figure 18: Dimensions of bridge showing sidewalks and center line	71
Figure 19: Girder layout.....	72
Figure 20: The layout of traffic flow during construction with the southbound lane of Rt. 122 closed	73
Figure 21: The layout of traffic flow during construction with the northbound land closed.....	74
Figure 22: Standard traffic patterns (U.S. Department of Transportation, 2007).....	76
Figure 23: Formulas for Tapers (U.S. Department of Transportation, 2007).....	77
Figure 24: Shows a detour to get around ramp closings (Google Maps).....	79
Figure 25: Detour of Eastbound off ramp of Rt. 20.....	80
Figure 26: Traffic Control Plan for Rt. 20 When the Eastbound Lane is Closed	82
Figure 27: Traffic Control Plan for Rt. 20 When the Westbound Lane is Closed.....	83
Figure 28: Lagging Being Used as a Protective Barrier Below I-290	85
Figure 29: Google Map Site Design for Storage	87
Figure 30: Areas for onsite Operations During Southbound Construction.....	88
Figure 31: Areas for onsite Operations During Northbound Construction.....	88
Figure 32: Picture of Rt.122 Bridge from Rt.20 (Google Earth).....	99
Figure 33: Arial view of the bridge and surrounding area (Google Earth).....	100
Figure 34: Construction Phase Flow Chart.....	102
Figure 35: Surrounding area showing storage locations (Google Earth).....	104
Figure 36: Construction Phase Methodology	105
Figure 37: Material options.....	106
Figure 38: Framework for material selection.....	106
Figure 39: Material Selection Methodology	108
Figure 40: LRFD Design Flow Chart	109
Figure 41: LRFD Design Methodology	110
Figure 42: Sustainability Flow Chart.....	111

Figure 43: Sustainability Methodology	112
Figure 44: Life-Cycle Cost Analysis Flow Chart	113
Figure 45: Life-Cycle Cost Analysis Methodology	114
Figure 46: Project Schedule	117

List of Tables

Table 1: Section properties	32
Table 2: General Information for Composite Deck Design.....	44
Table 3: RISA 2D Calculations for Un-factored Max Positive and Negative DL Moments	45
Table 4: Composite Deck Reinforcement.....	46
Table 5: Un-factored Moments due to Dead and Live Loads.....	50
Table 6: Un-factored Shear	50
Table 7: Pre-Stressed Frictional Loss	51
Table 8: Pre-Stressed Loss.....	51
Table 9:Pre-stress loss Equations.....	51
Table 10: Moment Coefficients	52
Table 11: Jacking Force	52
Table 12: Concrete Stress	53
Table 13: Initial Cost Unit Prices.....	55
Table 14: Inflation Rates.....	56
Table 15: Adjusted Unit Prices	57
Table 16: Spacing of Reinforcing Steel Used to Calculate Quantity of Galvanized Reinforcing Steel.....	58
Table 17: Bridge Dimensions Used to Calculate Various Quantities.....	58
Table 18: Total Initial Cost of Construction Materials.....	59
Table 19: Life-cycle repair frequencies	61
Table 20: Life-cycle Repairs.....	65
Table 21: Initial and Life-Cycle Cost	65

1. Introduction

Much of the United State's infrastructure was built during the Great Depression. Since, then, much of this vital infrastructure has not received the maintenance necessary in order to keep it in working order. In addition, many of these structures have outlived their expected life span and should be replaced.

Mass Highway has a number of different bridges and overpasses that are in need of new design and construction in the Worcester area. One of these bridges is a two-span overpass which is part of Grafton Street in Worcester. This overpass spans approximately 88 feet. Two designs were analyzed based on such factors as cost, constructability, life-span cost and maintenance. The existing bridge has two spans. Such design ideas as making it a one-span bridge and what material to use in construction, pre-stressed concrete or steel, were further investigated. Finally, a complete design of the bridge including girders, columns, abutments and the deck was created. Also, a cost estimate for materials was made using unit prices from various sources. A life-cycle cost analysis was also preformed. In these two procedures sustainable practices were looked at in an effort to reduce costs.

2. Background

Before we were able to start the design and methodology phase of our project, we had to research and identify the different components of a typical highway overpass. Along with the components of an overpass, we investigated the site design, constructability, cost influence, and the condition of the current structure.

2.1 Rt. 122 Highway Overpass

The subject bridge is the Rt.122 highway overpass which extends over Rt.20. It is located southeast of downtown Worcester, as can be seen in Figure 1. Because of its close proximity to the city and the Mass Pike, it is a very heavily trafficked road. It serves a wide variety of motorists from local traffic to tractor trailers. The bridge was built in 1931 during the Great Depression. Since then, it has not had a major rebuild or rehabilitation. Also, the bridge has exceeded its expected service life of 75 years and is due for replacement in the near future. The bridge is built out of very typical materials from that era. It has steel stringers and girders and a cast-in-place concrete deck.

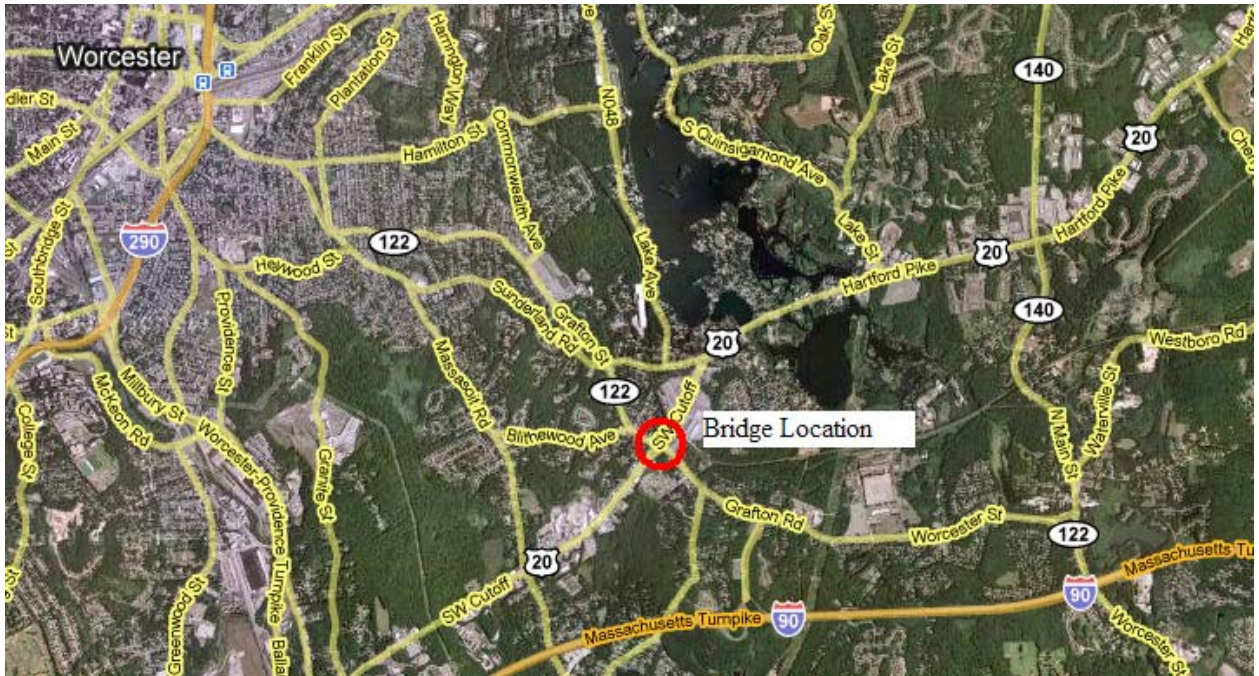


Figure 1: Aerial view of the bridge location (Google Earth)

2.1.1 Issues with the bridge

Due to the bridge's age, there are a number of structural issues that need to be corrected. These issues include minimal clearance, insufficient site distance, and the lack of drainage on Rt. 20 under the overpass. In addition to this, there are other problems that have to do with design. When this bridge was built in 1931, traffic patterns were much different. This bridge was built for smaller cars and trucks than it has to service now.

2.1.2 Structural Problems

The largest structural problem with this bridge is with the superstructure. It received a rating of 3 from a recent Mass Highway Inspection. This means that it is in serious condition. This superstructure has 13 steel girders running the length of it, and every aspect of these girders was rated satisfactory or worse. Major parts of the superstructure, from the welds, to the member alignment, to the diaphragms are classified as in serious condition. The last time

they were painted was in 1978, and the elapsed time has allowed a considerable amount of corrosion to occur. Also, there was a major collision when a tractor trailer hit the deck girders. This caused lots of bending in the supports.

Along with the superstructure, the condition of the substructure is also a concern. However, it was rated a little higher than the superstructure, at a 5, or fair rating. The substructure does not rely on piers; rather it is basically a shallow foundation. The walls, such as the backwall and breastwall are in fair condition. However the settlement is still in good condition.

2.1.3 . Design Problems

There are three major design problems which Mass Highway stated need to be addressed in the rebuilding of this bridge. They are site visibility, clearance, and drainage. Site visibility is a very important part of any bridge. It contributes to safety and allows drivers to see what is on the other side of a bridge. However, the Rt. 122 highway overpass has a 6 ft reinforced concrete center pier which hinders this, as can be seen in Figure 2. To increase site visibility, Mass Highway wants to remove this center pier.



Figure 2: Center reinforced concrete pier (Google Maps)

Accomplishing this goal could be very easy if there wasn't already a problem with clearance. Taking out the center pier would cause there to be a longer clear span. When calculating the forces acting on the supports, the load is multiplied by the length of the span. Therefore, the longer the clear span, the greater the forces acting on the members, and the deeper the members would have to be in order to support the load.

This bridge has an actual clearance of less than 14.5 ft. However, on highway overpasses, the absolute minimum clearance allowed is now 14.5ft and ideally is 16.5 ft (Cox, 2008). This lack of vertical clearance has already led to one severe accident on the eastbound lane of Rt. 20, The damage to the girders can be seen in Figure 3.



Figure 3: Damage From Collision

Thus, the redesign of the bridge should result in a larger vertical clearance. This will be made difficult by the fact that the bridge is going to have a longer effective span once the center pier is removed. Also, the height of the bridge can only be raised by 6 inches to compensate for the additional member depth. This is because there are many businesses and roads leading off of Rt. 122 near the bridge. If the bridge was raised over 6 inches, the approaches to the bridge

would have to have long slopes, which would adversely affect the driveways of these local businesses.

In addition to visibility and vertical clearance, there is currently a problem with water pooling underneath the bridge. If this problem is ignored, it could eventually lead to the undermining of the bridge's foundation. The pooling of water also proves that Rt. 20 dips under the bridge. Lowering the level of Rt. 20 would ultimately help add clearance, but would have an adverse affect on the pooling issue.

2.2 Overview of Bridge Components

In general, all bridges are separated into a superstructure and a substructure. Figure 4 below illustrates these two parts. The superstructure is defined as all portions of the bridge above the substructure (Tonias, 1995). The function of the super structure is to collect the live loads and concentrate them into the substructure. The main components of the superstructure are the wearing surface, the deck, the primary members, and the secondary members. This is the most visible portion of the bridge. The substructure acts as a foundation to the bridge. It is comprised of the abutments, piers, bearings, pedestals, and retaining walls.

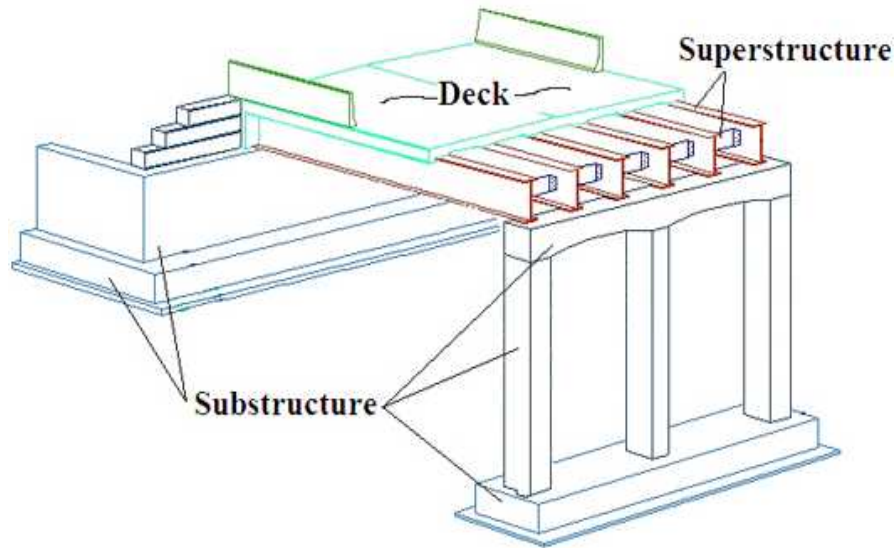


Figure 4: Superstructure and Substructure (Oklahoma Bridge Tracker, 2009)

2.3 Wearing Surface

When traveling over any bridge, the most visible portion is its wearing surface. The wearing surface is generally made of bituminous concrete or asphalt. It is exposed to all traffic travelling across the bridge. It is also exposed to the weather. Snowfall is common during the winter months in New England. The de-icing chemicals used to make the roads safe degrade the wearing surface of the bridge. Over time, the wearing surface becomes increasingly damaged by the elements and has to be repaired or completely resurfaced. Generally, this layer of bituminous concrete is between 2 and 4 inches thick. Due to the constant repair and resurfacing, this thickness generally increases over time. (Tobias, 1995)

2.4 Bridge Deck

The bridge deck sits directly below the wearing surface. It is what supports the wearing surface. The bridge deck is generally made of a reinforced concrete slab or a large steel plate. The purpose of the bridge deck is to distribute the loads transversely. It distributes the loads

along the bridge to the underlying structural elements, such as the girders and stringers. The deck is generally directly connected to the supporting girders/stringers, or is separated by a steel plate which connects the two together. (Tonias, 1995)

2.5 Primary Members

The primary members of the bridge are responsible for distributing the loads from the bridge deck longitudinally. The primary members are the girders that run below the bridge deck. These girders are typically made of structural steel or concrete. The most common types of steel girder are the rolled beam and the rolled beam with cover plates. (Tonias, 1995)

2.5.1 Rolled Beam

Figure 5 below shows an example of a typical hot rolled steel I-beam. This type of steel is steel that is rolled to its size while still very hot, over 1700 °F. Because of this, the size of the steel isn't always as precise as with cold rolled steel. However, it is still very commonly used in highway bridge design. This is because it is much less expensive than cold rolled steel. Along with this, it has many other positive attributes, such as: it comes in many different sizes and shapes, is easy to assemble on site by welding or bolting, allows for a lot of offsite fabrication, is possible to recycle and use recycled steel, and is very strong in tension.

However, hot rolled steel also has some negative characteristics. One major drawback is that hot rolled steel, like all steel beams can rust. This can be prevented with rust proof coatings, but this has to be maintained otherwise the life-span of the bridge will shorten. Another drawback of using steel girders is site design and storage. The steel girders have to be shipped to the site from a fabrication plant. At the site there has to be an ability to work

with them before they are able to be put in place. This would entail a space to move girders close to the site and an area for a crane to move them.

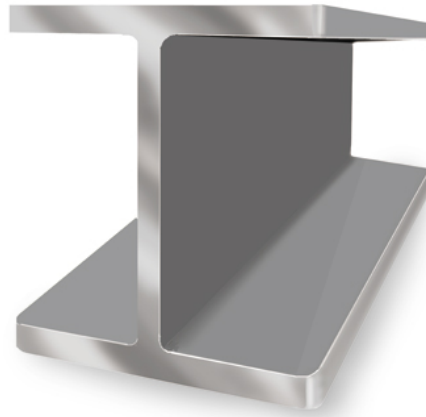


Figure 5: Rolled Steel I-Beams (Foam Laminates of Vermont, 2008)

2.5.2 Pre-stressed Concrete Girders

Over the past fifty years, pre-stressed concrete bridges have become the most popular structural system for bridge design, because of their high strength, low life-cycle cost, and efficient assembly (pcap.org/Presentations/files/acs.ppt). There are two main types of pre-stressing systems. The first is a pre-tensioning system where the steel strands are tensioned before the concrete is placed. The second type of pre-stressing system is a post-tensioning system. In this case, the steel is not tensioned until after the concrete has been placed and has gained sufficient strength. Due to the fact that it is a combination of both concrete and steel, pre-stressed concrete has both high tensile and compressive strengths.

Many case studies have been done involving the use of pre-stressed concrete girders in the design of long, continuous spans. In some cases, pre-stressed concrete has

been used in spans up to 150 feet in length. One of the main factors in our project is the clearance issue. Pre-stressed concrete girders are able to span long distances while not giving up much depth. Below is an illustration of a bridge in Denver, Colorado that spans 80 feet and runs over a busy urban road. This bridge contains similar traffic patterns to our bridge and has a total girder depth of 3 feet, which includes the 3 inch thick wearing surface. In contrast, the existing bridge has over 3 feet in girder depth plus an additional 8 inch wearing surface. Therefore, it is possible to use pre-stressed concrete in long spans without giving up strength or depth.



Figure 6: Pre-stressed Concrete Bridge in Denver, Colorado (Colorado DOT, 2009)

2.5.3 Steel-Concrete Composite Box Girders

Another construction system that is often used in highway bridge design is steel-concrete composite box girders. These primary members have many advantages ranging from their flexural capacity to torsion resistance. Also, they provide a closed system which limits the exposure of the primary members to the elements. In turn, this limits the life-cycle costs by reducing corrosion. Finally, they are aesthetically pleasing and have the capacity for longer/wider spans.

There are two design methods when using box girders. The first is to have one large box girder running along the entire span. This box girder can either have a single cell or multiple cells within its cross section. The single box girder consists of two webs, a bottom flange and two top flanges. The multiple webs share shear forces and reduce shear lag (Duan, 1995). The other design is to have two or more box girders extending the length of the span. In a multi-box design, the boxes are generally smaller and closer together than a single box system. This causes both the torsion and flexural stiffness to be greater than a single box system.

Figure 7 below shows a typical multi-cell box girder system. Although it is possible to use a multi-box design, it has been proven to be more efficient and economical to use a single box design with a larger cross section (depth-to-span ratio = 0.4). However, having a large cross section could cause issues with our bridge. There is already a clearance issue and if there was a large cross section for the primary members, they would be deeper, thus making the clearance less (Duan, 1995).

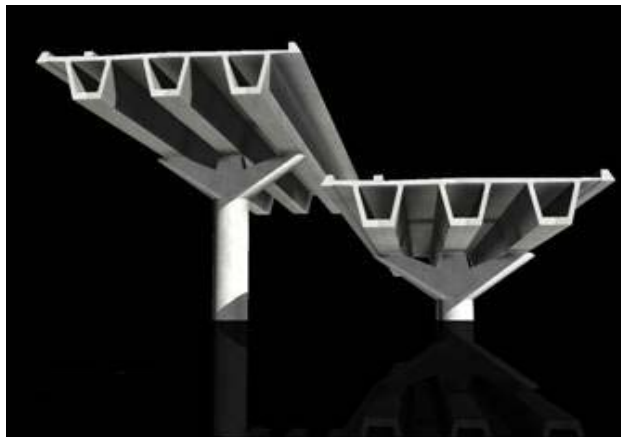


Figure 7: Multi-cell Box girder System (New Crossing, 2008)

2.6 Secondary Members

The secondary members act as bracing for the primary members. They run perpendicular to the primary members. This prevents lateral movement and they can prevent tensional forces (Tonias, 1995).

2.7 Abutments

The abutments are a part of the substructure or foundation of the bridge. They act as end supports. Abutments provide vertical support to the bridge and lateral support to the soil at the ends of the roadway. (Tonias, 1995) There are ten types of abutments. There is a Stub Abutment, a Full-Height Abutment, a Gravity Abutment, a U Abutment, a Cantilever Abutment, a Semi-Stub Abutment, a Counterfort Abutment, a Spill-through Abutment, a Pile Bent Abutment, and Reinforced Earth Systems. The bridge being analyzed has Full-Height Abutments.

A Stub Abutment is found at the top of an embankment. Due to this fact, they are usually supported by piles that are driven into the ground (AASHTO, 1987). Although they can also be found lying on gravel or even the ground itself, stub abutments generally need the extra support of the piles. This is especially true in New England. Most soil in New England can be classified as Type C soil. This means that a large portion of the soil is composed of loose rock or sand particles. This type of soil is not nearly as supportive as a Type A soil, which has a higher bearing capacity than Type C soil. An example of Type A soil would be clay.

The second type of abutment is known as the Full-Height Abutment. This type is much larger than a stub abutment. The Full-Height Abutment is a cantilever abutment that runs

from the roadway below all the way up to the roadway surface overhead. It is basically a giant wall that supports the overlying roadway at its ends.

Gravity Abutments are very common and are typically made out of concrete or stone masonry. They consist of a backwall and flaring wingwalls, all of which sit on top of a footing. This type of abutment uses its weight in order to resist the horizontal earth forces. The fact that this type of abutment is so heavy is the reason for the necessity of the footings. (Tonias, 1995)

The fourth type of abutment is the U Abutment. The U Abutment is very similar to the Gravity Abutment. It too is made of a backwall and two wingwalls. The main difference between the two is that the wingwalls on the U Abutment are attached to the backwall at right angles. Finally, the wingwalls of a U Abutment can vary in thickness. For example, the thickest section of the wingwall is generally found where it is attached to the backwall. The thinnest section is typically found on the free end of the wingwall. (Tonias, 1995)

Cantilever Abutments are abutments that are attached to footings and extend upward. A Cantilever Abutment has a bridge seat and is able to resist large vertical loadings. Much like the Gravity Abutment, the dead weight of the Cantilever Abutment is used, along with the footings, to resist the horizontal earth loads. Typically, this type of abutment is used for heights up to 21 feet. For any heights greater than this, a Counterfort Abutment would be used. (Tobias, 1995)

A Counterfort Abutment uses a stem and a footing, which are braced by slabs known as counterforts. This allows the abutment to act as a horizontal beam between each counterfort (spaced along the footing). This is unlike the Cantilever Abutment which is only attached to

the footings and acts as a cantilever stem. This fact allows the Counterfort Abutment to be used for large heights that extend upwards of 21 feet. (Tonias, 1995)

Another type of abutment is known as a Spill-through Abutment. Unlike the other abutments mentioned, the Spill-through Abutment is not a wall, but rather two columns with a cap beam resting on top. The cap beam is responsible for supporting the bridge seat, which in turn supports the superstructure of the bridge. Due to the fact that there is a gap between the two columns, only a fraction of the embankment is supported by the abutment. Soil from the embankment is able to spill through the gap between the two columns. A Pile Bent Abutment is identical to a spill-through abutment; it is distinguished by the fact that it is supported by one or two piles rather than columns. (Tonias, 1995)

2.8 Piers

The existing bridge is comprised of two equal spans with a six foot wide pier in the middle. Although the objective is to remove the center pier in order to reach project goals, there is a chance that this may not be possible.

“A pier is a structure located at the end of a bridge span which provides the basic function of supporting spans at intermediate points between end supports (abutments)”. (Tonias, 1995) Piers have three main functions which are to carry their own weight, support the dead and live loads provided by the superstructure, and to transmit all loads to the foundation of the bridge or overpass.

Like abutments, there are a number of different types of piers. Selection of which type of pier/column to use is based on aesthetics, shape of the superstructure, and the fact that the pier/column should provide limited interference to passing traffic. There are six different types

of piers. They include hammerhead, column bent, pile bent, solid wall, integral, and single column. As previously stated, the use of each type of pier is used based on different criteria (Tonias, 1995).

In the case of the Rt. 122 overpass, a solid wall pier is used. “A solid wall pier (also known as a continuous wall pier) as its name would imply, consists of a solid wall which extends up from a foundation consisting of a footing or piles”. (Tonias, 1995) It is not recommended to use this type of pier to support a wide superstructure because it causes sight distance issues for traffic traveling under the bridge or overpass. If a solid wall pier is used for wide superstructures, proper lighting is necessary. In the case of the Rt. 122 overpass which is over 70 feet wide, a solid wall pier is used without sufficient lighting. As previously stated, this causes a sight distance issue, which we planned to eliminate with a new and more effective design of this overpass.

2.9 Bearings

Bearings may be a small portion of any bridge or overpass, but their importance cannot be overlooked. The main function of a bearing is to transmit loads from the superstructure to the substructure. There are two main categories of bearings, fixed bearings and expansion bearings. Fixed bearings allow for rotation at the member's end and resist translation. On the other hand, expansion bearings allow both rotation and translation. These types of movements occur due to creep, shrinkage, settlement, uplift, loading, and thermal forces. These bearings are also exposed to various types of loading which include the dead load of the superstructure, traffic live loads, wind loads, and seismic loads.

Within the two categories of bearings, there are several different types. These are rocker bearings, roller bearings, sliding plate bearings, pot bearings, spherical bearings, elastomeric bearings, and lead rubber bearings. Although there are seven different types of bearings, for the purpose of this specific overpass and its loadings/dimensions, we chose to focus on three: roller bearings, rocker bearings, and elastomeric bearings.

This section was an important first step in completing this project. It provided information pertaining to the different components of a highway overpass. Along with providing information about the overpass it also gave us insight on other parts of our project, such as the construction phase and cost analysis. This was important because it helped us to determine what our methodology would entail.

3. Methodology

Our group focused on developing a new bridge design to solve the three major problems with the current bridge. These are the low clearance of the bridge, water pooling, and the sight-distance issue that is caused by the 6 foot wide center pier. In order to completely solve each problem the first step our group took was to consider solution alternatives that took out the center pier while gaining clearance. After material research and discussions with a representative from Mass Highway, we chose to investigate the possibility of using a composite deck-girder design and a pre-stressed concrete design (Nabulsi, 2009). In both of these systems, the steel and concrete work together to limit the effects of the loadings. Next we looked at the cost of the bridge. This cost included both the initial and life-cycle cost. Sustainable techniques, such as using recycled materials, were implemented in an effort to reduce cost. Finally, we looked at the construction phase. The construction phase included traffic control, material selection for the primary members, and site design for construction operations.

3.1 LRFD Design

A number of factors were considered when choosing which designs to attempt. First, our group took into account the constructability of the design. Some designs are easier to erect due to their pre-fabrication. We also considered the impact that the designs would have on the traffic flow. For example, it would be beneficial to have a design that could be constructed piece by piece. This would ultimately allow us to keep half of the bridge open at a time which would have limited impact on traffic flow. Finally, after discussions with a designer from Mass Highway and

researching the properties of typical building materials, our group decided to attempt two different designs. These designs were a composite deck and steel girder system and a pre-stressed concrete system. Each design would allow us to erect the bridge in sections. Also, pre-stressed concrete sections could be pre-fabricated which would make the erection process simple and quick. In each of these, we followed the AASHTO design guidelines.

3.1.1 Composite Deck Design

The first design that we investigated was a composite concrete deck and girder system. Clearance was an issue with this bridge along with sight distance. Our group felt that this design could be a possible solution to these problems. A composite system supports loads more efficiently than a conventional cast-in-place concrete deck design because the deck works together with the girders. In full, the deck supports most of the load in compression while the steel girders support most of the load in tension.

The first step in designing a composite concrete deck, similar to the first step in all deck designs, is to obtain design criteria. Design criteria outline the method used in the design along with the known values that will be used in subsequent steps. We used the equivalent strip method to design the composite concrete deck. This method is based on the idea that a transverse strip supports the truck axle loads. This strip is also assumed to be located at the center of the girders. According to AASHTO specifications, design factored moments and shear values are calculated using coefficients known as load factors. Finally, shear and fatigue in the reinforcement do not need to be investigated when using the equivalent strip method. This is because this specific method is only an approximation, unlike the empirical method which utilizes laboratory testing to ensure the design is appropriate. Design criteria for this specific deck design included deck

properties such as girder spacing, number of girders, deck top cover, deck bottom cover, density of concrete, concrete 28 day compressive strength, reinforcing steel strength, and density of the future wearing surface. The design criteria also included parapet properties such as weight per foot of the parapet, width of its base, moment capacity at base, parapet height, critical length of yield line failure pattern, and total transverse resistance of the parapet. All hand calculations can be seen in Appendix B.

More values had to be looked up in the *AASHTO Specifications Manual* before the actual design process began. First, we had to decide on the thickness of the deck. According to AASHTO, S9.7.1.1, the minimum slab thickness is 7 in. In order to be conservative, and also due to the weather in New England (deicing salts), we chose a slab thickness of 8 in. Also, we had to determine the overhang thickness. This is vital because the overhang is the portion of the slab that supports the parapets. The minimum overhang thickness denoted by AASHTO was 8 in; therefore we chose a thickness of 9 in. to be used in our design.

The next step was to analyze the live and dead load affects. After this was done, the following steps were taken in order to complete the design of the composite concrete deck:

- Design for Positive Flexure in Deck
- Check for Positive Flexure Cracking under Service Limit State
- Design for Negative Flexure in Deck
- Check for Negative Flexure Cracking under Service Limit State
- Design for Flexure in Deck Overhang
- Check for Cracking in Overhang under Service Limit State
- Compute Overhang Cut-off Length Requirement

- Compute Overhang Development Length
- Design Bottom Longitudinal Distribution Reinforcement
- Design Top Longitudinal Distribution Reinforcement
- Draw Schematic of Final Concrete Deck Design
- Establish entire cross-section including the deck, pavement and parapets

3.1.2 Composite Girder Design

We chose the method of composite girder design because the composite system increases the effective strength of the girders, thus allowing them to be smaller, and hopefully fix the clearance issue. In this method, we used $\frac{3}{4}$ " diameter studs, 50 ksi steel, and concrete with a strength of 4 ksi.

The first step in designing the composite girders was determining the loading types and calculating the loads. The steel girders are simply supported members with ends that do not prohibit rotation. We looked at the dead load of the bridge itself, wearing surface, and the parapet placed on the outside of the bridge. We also looked at live loading. For live loading, we used HL-93. This is the standard AASHTO design for highways. Although our bridge is only going to have 2 lanes of traffic, each about 27' (10m) wide, we chose to calculate the live load as if the whole bridge had 11' (3.66 meter) wide lanes. This is because 11' is the standard size and we want to make sure that if part of the bridge ever has to be closed and traffic gets condensed, the bridge will be able to support the load. We used the same loading for both composite girder and pre-stressed systems.

Next we had to select the steel beam for composite action. One of the first steps of this was to calculate the live and dead loads acting on the bridge. We also had to determine exactly where in the composite beam it goes from compression to tension. Once this was done, we looked at the *AISC Steel Construction Manual* to determine which beam would work. After this we calculated the number of studs required by looking at the shear strength of each stud and the total longitudinal force to be transferred between the concrete slab and steel girders.

Once we chose our steel beam, we checked to make sure it was strong enough to hold before the concrete hardened. This was done by seeing the force applied to the beam by the wet concrete, and service live load. Along with making sure the beam could hold the weight, we looked to make sure the deflection would not be too great. The deflection was compared to the beam's length to check this (the limit was $L/360$). We also checked to make sure that the service LL deflection wouldn't be too high once the concrete had hardened (again the limit was $L/360$). The last thing we did was check the shear in the beam. The capacity had to be larger than the shear load applied to it. All hand calculations can be seen in Appendix C.

3.1.3 Pre-Stressed Concrete

When we focused on pre-stressed concrete, we first had to determine what type of structure to use. There are many different types, I beams (types I-VI), solid beams, t-beams, and boxes. We chose to use a cast-in-place (CIP) box beam. This was because box beams have relatively large moments of inertia compared to height and offer the best chance in meeting the clearance goals. For our CIP box, we used standard property values, outlined in Table 1.

Table 1: Section properties

<u>Material</u>		
Initial Concrete	$f'_{ci} = 24 \text{ MPa}$	$E_{ci} = 24,768 \text{ MPa}$
Final concrete	$f'_c = 28 \text{ MPa}$	$E_c = 26,752 \text{ MPa}$
	$f_{pu} = 1860$	$E_p = 197,000$
Pre-stressing steel	MPa	MPa
Mild steel	$f_y = 400 \text{ MPa}$	$E_s = 200,000 \text{ MPa}$
<u>Pre-stressing steel</u>		
Anchorage set		
thickness	10 mm	
Pre-stressing		
@jacking	$f_{pj} = 1488 \text{ MPa}$	

The first step was to determine the cross sectional properties. First, we decided that the bridge's cross section should be about 35 inches deep from the top of the roadway surface to the bottom of the supporting girders. This is because the current bridge is 41" deep and an additional 6" inches of clearance was desired. So, we made the bridge .889 m (35 inches) deep including the road surface. Next, we determined the distance between each girder. We decided that the distance between each exterior girder would be 20.5m, this accounts for 1.2m on each side as an overhang. Then, knowing that the distance in between each girder should be less than 2 times their depth, we decided to have 1.7 meters in between each one. This would mean that there are 13 girders. All other dimensions are shown below in Figure 8.

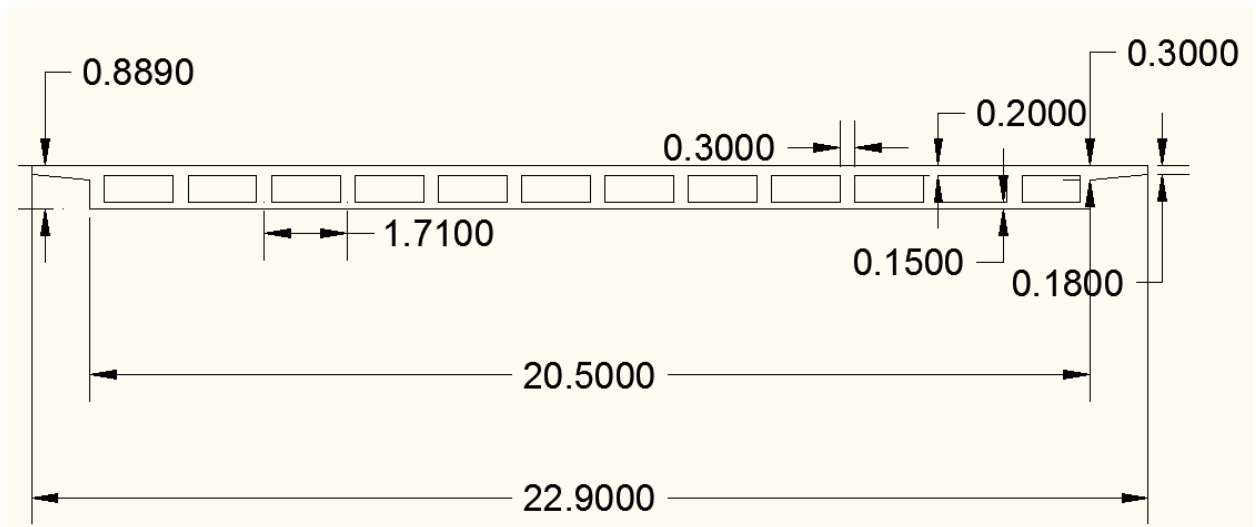


Figure 8: Cross-Sectional Diagram

Once we had the bridge properties determined, we started to follow the AASHTO design guide in creating the bridge. The hand calculations can be seen in Appendix D. The first step was to calculate the loads. This was the same as the composite girder system. We used HL-93 live loading with 3.66m lanes. After the loads were calculated, we computed the live load distribution factors. Next, we determined the moment and shear forces. The loadings we used can be seen in Figure 9 below.

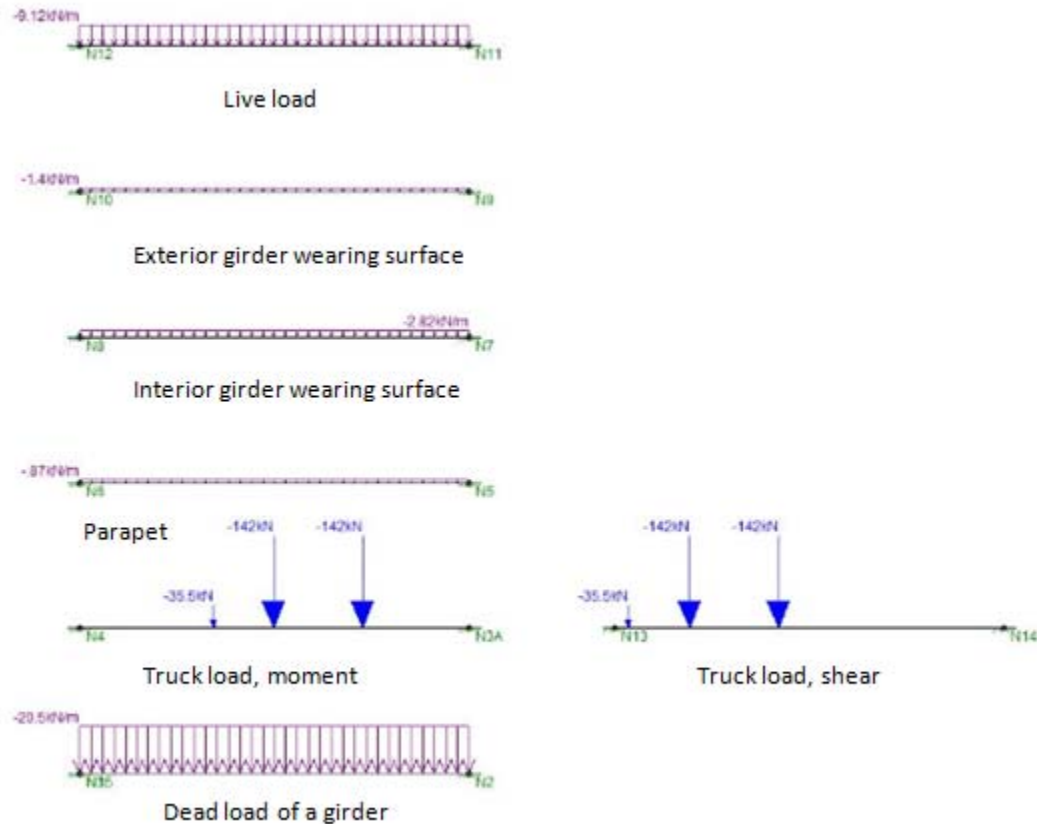


Figure 9: Loading

For the design of the bridge, we used fixed ends that prevented rotation. Because of the fixed ends, we used RISA for pre-stressed concrete. We also looked at loading cases of strength limit state I and service limit state I. Once this was done, we followed the standard AASHTO design method. This is outlined below.

- Calculate pre-stress losses
 - Frictional losses
 - Anchorage set losses
 - Elastic shortening losses
 - Time-dependent losses
 - Total loss

- Determine pre-stressing force for interior girder
 - Force coefficients
 - Moment coefficients
 - Stresses in bottom and top fiber
 - Determines required area of pre-stressing steel
- Check concrete strength for interior girder, service limit state 1
 - Concrete stresses after instantaneous losses
- Flexural strength design for interior girder, strength limit state 1
 - Concrete strength after total losses
 - Factored moments for an interior girder
- Shear strength design for interior girder, strength limit state 1
 - Flexural strength design
 - Factored shear
 - Shear strength design
 - Steel required for shear

Using a fixed end cast-in-place girder system will cause us to perform an extra task of redesigning the abutments in order to resist the moments caused by the girders. But, using fixed ends would lessen the mid-span moments and deflections. This would then allow us to use a shallower girder. The design of the abutments would include adequate rebar which would resist the rotational forces. However, we found that the force generated by the pre-stressing steel was too high (124 Mpa) for the concrete to resist, thus eliminating this design as an option. Therefore, there was no need to redesign the abutments for this system.

The design phase was a crucial section of our project. This is because meeting Mass Highway's goals such as increasing sight distance and clearance was our main priority. Other factors such as cost and constructability were addressed to make the final decision on the design to be used.

3.2 Cost Analysis

In determining the cost of any large building project there are two things which have to be considered, the initial cost and the life-cycle cost. The initial cost is the cost of the construction of the bridge. This includes demolition of the current bridge, materials and components for the new bridge, and construction fees. This is something that gets bid on by different contractors who all submit different proposals. Along with initial cost, it is important to look at life-cycle cost. Large structures have a high amount of maintenance. We estimated how much the maintenance on the bridge would be in its expected service life.

3.2.1 Initial Cost

Our group estimated the original cost of the bridge. To obtain initial cost prices, we first gathered unit prices from a number of sources. Our main source was the *2009 R.S. Means Index*. We also looked at the Florida DOT website and contacted subcontractors in Massachusetts. Our group adjusted all prices based on average inflation rates. Also, for the unit prices obtained from the Florida DOT website, we used a location factor to determine the price for Massachusetts.

After gathering unit prices for all the materials, we measured the quantities for each material. In order to do this, we had to refer to our designs for various dimensions. After this, we multiplied our quantities with the unit prices to determine the final result. All calculations for take-off were done using an Excel spreadsheet.

Finally, our group analyzed ways to lessen the initial environmental impact of the bridge through the use of recycled materials. We researched the salvage value for both concrete and steel. We also researched the cost of using recycled materials in the construction of the new bridge and whether or not there were any structural benefits of using recycled materials. From

our research, we were able to make recommendations according to the cost benefits of using recycled building materials.

3.2.2 Life -Cycle Cost

In determining the life-cycle cost of the new bridge to be built, there were 3 main steps. The first step was to determine the expected life of the bridge. The second step was to determine what has to be done to the bridge in its lifetime to keep it operational, and the third step was to calculate how much each one of those things would cost. The first step was very straight forward. In order to complete this, we looked at standard bridge expectations from Mass Highway and found out how long they expect each bridge to be operational. We also spoke with a Mass Highway designer to get a better insight on the life-span of a bridge in New England.

Once this was done, we researched the maintenance operations that had to be done during the life-span of the bridge. This also included the affects of implementing sustainable practices such as solar powered bridge lights and water control. We looked at the frequency of these maintenance operations and how much they cost.

From there, our group developed a spreadsheet that calculated the present worth of future required maintenance. The costs spreadsheet took inflation into account, along with an assumed discount rate of 6% annually. In this spreadsheet we determined how much money would need to be invested now, at the 6% discount rate, for all future maintenance to be accounted for. We then compared the invested amount to the total cost of the maintenance procedures during the life-span of the bridge.

3.3 Construction Phase

The construction phase was another major aspect of our project. We separated the construction phase into three sections. The first was traffic control. Our group had to devise a traffic control plan for both Rt. 122 and Rt. 20 that would be utilized during demolition and construction. The second section was site-design. Our group analyzed the site for storage space and access for construction operations. Storage space is vital for any construction site. Not only must we store the building materials, but also the heavy machinery that will be used. The final section was material selection, influencing this was ease of construction, weather conditions, along with other factors. Our group reviewed the different types of materials that were used in the new design. We focused on the most efficient way to dismantle the old structure and erect the new.

3.3.1 Traffic Control

From the start of this project, we knew that one of the largest issues in the site design for this bridge was going to be traffic control. This is because both roads are very busy. Rt. 20 is a major industrial road with heavy traffic. Together, with Rt. 122, they are pivotal in linking Worcester to the Mass Pike and surrounding areas. In addition to this, where these two roads cross is an extremely urbanized area making site access and operations much more difficult.

Health and safety was a main focus of this project. The area where this is most greatly affected is in traffic control. Last year, over 1,010 people died due to motor vehicle crashes in work zones in the United States (Mass Highway, 2008). This is something that could be prevented through a good Traffic Management Plan (TMP). In designing our Traffic Management Plan we considered (Washington State Department of Transportation, 2006):

- The safety of pedestrians, bicyclists, and motorists traveling through the work zone;
- Protection of work crews from hazards associated with moving Traffic;
- Capacity of facilities and delays to users;
- Maintenance of access to adjoining properties; and
- Issues that may result in project delay.

Adding to this already difficult situation, we realized that there were going to be issues with both the roads. Initially we thought that just Rt. 122 was going to have to be closed or detoured. However, after talking to a Project Engineer at the Mass Highway District 3 Office, we learned that when dismantling a bridge, we also had to look at the bottom road and take necessary precautions (Nabulsi, 2009).

3.3.1.1 Route 122

The first part of traffic control that we looked at was what to do with Rt. 122. This is a very busy road with daily traffic of 24,400; and 15% of which are trucks (Massachusetts Highway Department , 2009). We felt like there were going to be the most issues with this road because it has the bridge which is going to be replaced. We found that there were three possible options for traffic control. These are re-routing traffic completely around the bridge, only closing part of the bridge at a time and allowing limited traffic, or installing a temporary bridge. Our first step was to determine which method to use to control traffic flow. Once we chose a design for

traffic flow, our group analyzed the feasibility of the design for the construction phase. Finally, we designed the traffic plan for Rt. 122 on CAD while taking into consideration the site layout.

3.3.1.2 Rt. 20

Our group also had to devise a traffic control plan for Rt. 20 which runs underneath the overpass. In order to do so, we researched different methods that are generally used in such construction projects. We limited our search to two methods. The first involved closing either Rt. 20 eastbound or Rt. 20 westbound while the bridge was being demolished. Rt. 20 has two lanes running in each direction. Closing one side would still allow two-way traffic. The second method did not involve closing any roadways. Instead, a structure would be built underneath the bridge deck. The function of the structure was to collect any falling debris which results from demolition of the deck.

We analyzed each method to see which would work best for our planned construction process and Rt. 122 traffic plan. We also met with the lead Project Manager at JH Lynch (General Contractor). After our meeting and collected research, our group was able to devise an efficient and effective traffic control plan for Rt. 20.

3.3.2 Site Design

Another factor in the construction phase was site design. Both Rt. 122 and Rt. 20 are highly congested roadways which made this process more complex. In our site layout, our group focused on available storage space for both materials and access requirements for storage and operating heavy machinery. This was done by visiting the site and mapping out areas with sufficient space for storage. We had to also consider the distance of these areas from the actual

bridge. The closer the areas were to the construction site, the more efficient the process would be. Finally, we used Google Earth to illustrate which areas could be used to store the building materials and heavy machinery.

4. Results

This section includes the final LRFD design of the Rt. 122 overpass, an initial and life-cycle cost analysis, and details pertaining to the construction phase. These details include a traffic control plan during demolition and construction and site management for storage and operations. Our group was planning on analyzing the affect that different building materials would have on the construction phase and costs of the bridge. However, as will be seen in the LRFD results, only one material was able to meet our goals of increasing clearance and sight distance. Therefore, the sections following the LRFD results were specifically aimed towards one design instead of being the deciding factor in the material selection process.

4.1 LRFD Results

In designing the actual members of the bridge we followed the AASHTO specifications for the LRFD design method. We also used RISA 2D in order to calculate the member forces due to the applied loads. The results of this analysis were important because they affected many different parts of the project, including costs and material selection. We looked at 2 different types of designs: a composite deck and girder system and a pre-stressed concrete bridge.

The composite deck and girder system was the first system we looked at. This system includes a reinforced concrete deck which is connected to steel girders by shear studs. This connection allows the concrete deck to work with the steel girders to resist bending. Due to the material properties of both concrete and steel, the deck absorbs most of the compression forces while the girders absorb most of the tensile forces.

4.1.1 Composite Deck

The first step in designing the composite deck was to obtain the dimensions of the bridge and collect the general specifications; these dimensions can be seen below in Figure 10. This information was obtained through a number of resources including the Mass Highway Department, AASHTO specifications, and the Federal Highway Association website. The general information used in the design of the deck is illustrated in Table 2 below.

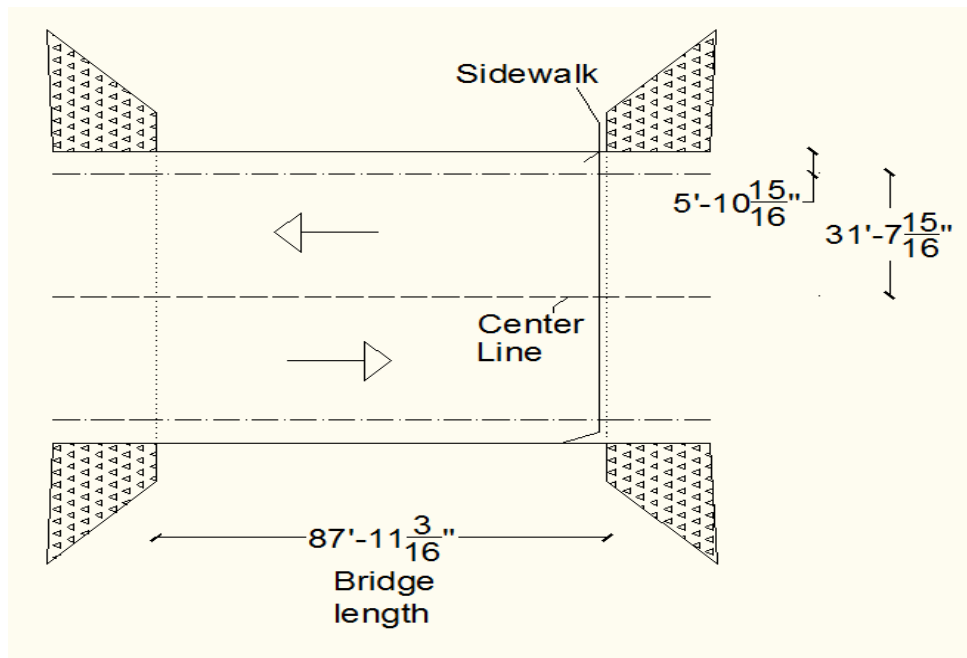


Figure 10: Dimensions of Rt. 122 Overpass

Table 2: General Information for Composite Deck Design

Bridge Deck	Values	Reference
Design Method	LRFD	
Deck Width	75.13 ft.	Mass Highway
Bridge Length	87.93 ft.	Mass Highway
Top Cover	2.5 in.	Mass Highway
Bottom Cover	1 in.	AASHTO Specs.
Structural Steel Yield Strength	50 ksi	AASHTO Specs.
Structural Steel Tensile Strength	65 ksi	AASHTO Specs.
Concrete Density	150 pcf	AASHTO Specs.
Concrete 28 day Compressive Strength	4 ksi	AASHTO Specs.
Reinforcement Strength	60 ksi	AASHTO Specs.
Wearing Surface Density	140 pcf	AASHTO Specs.
Deck Thickness	8 in.	AASHTO Specs.
Overhang Thickness	9 in.	AASHTO Specs.
Type F Parapet		
Mass Per Unit Length	650 lb/ft.	FHWA Design Example
Width at Base	1.69 ft.	FHWA Design Example
Moment Capacity at Base	17.83 k-ft./ft.	FHWA Design Example
Height	42 in.	FHWA Design Example
Length of Parapet Failure Mechanism (Lc)	235.2 in.	FHWA Design Example
Collision Load Capacity	137.22 kips	FHWA Design Example

The next step in the design process was to obtain design factors for both the live load and dead load moments for use in load combinations. These factors were obtained from different tables in the *AASHTO specifications manual*. For a two-lane bridge with girders spaced at 6 ft. on center, the live load factor was 1.75 (S3.4.1). There was also a dead load factor for the slab equal to 1.25, a dead load factor for the Type F parapet equal to 1.25, and a dead load factor for the future wearing surface equal to 1.50 (S3.4.1). These load factors were used to determine the

overall factored design moment for positive and negative moment areas. The load factors were multiplied with the un-factored positive and negative moments. Along with looking at the dead loads moments, which are shown in Table 3 below, we used the AASHTO guidelines for HL-93 live loading in calculating the moments for the deck. For the maximum positive live load moment, we used 4.83k-ft. For the maximum negative live load moment, we used -3.5 k-ft.

Table 3: RISA 2D Calculations for Un-factored Max Positive and Negative DL Moments

Member	Section	Moment (k-ft)
Deck Slab	1	-3.1
	2	0.388
	3	1.55
	4	0.388
	5	-3.1
Parapet	1	-2.077
	2	0.26
	3	1.038
	4	0.26
	5	-2.077
Future Wearing Surface	1	0.927
	2	-0.116
	3	-0.464
	4	-0.116
	5	0.927

When the maximum factored positive moments for live and dead loads were added together it came out to 12.44 k-ft. When the maximum factored negative moments were added together it came out to -11.47 k-ft.

The next step was to determine the proper size and amount of rebar needed for reinforcement in the bridge deck. There were various sections that we had to design for which included positive moment, negative moment, bottom longitudinal, top longitudinal and

overhang areas. When solving for the size and amount of rebar necessary, we had to check for parameters such as over-reinforcement and cracking under the service limit state. In the end, our results for reinforcement can be seen in Table 4 and Figure 11 below. All load calculations and design checks can be found in Appendix B.

Table 4: Composite Deck Reinforcement

Positive Moment	# 5 bars @ 7"
Negative Moment	# 5 bars bundled with #3 bars @ 4"
Bottom Longitudinal	# 5 bars @ 7"
Top Longitudinal	# 4 bars @ 10"
Overhang	# 5 bars bundled with #3 bars @ 4"

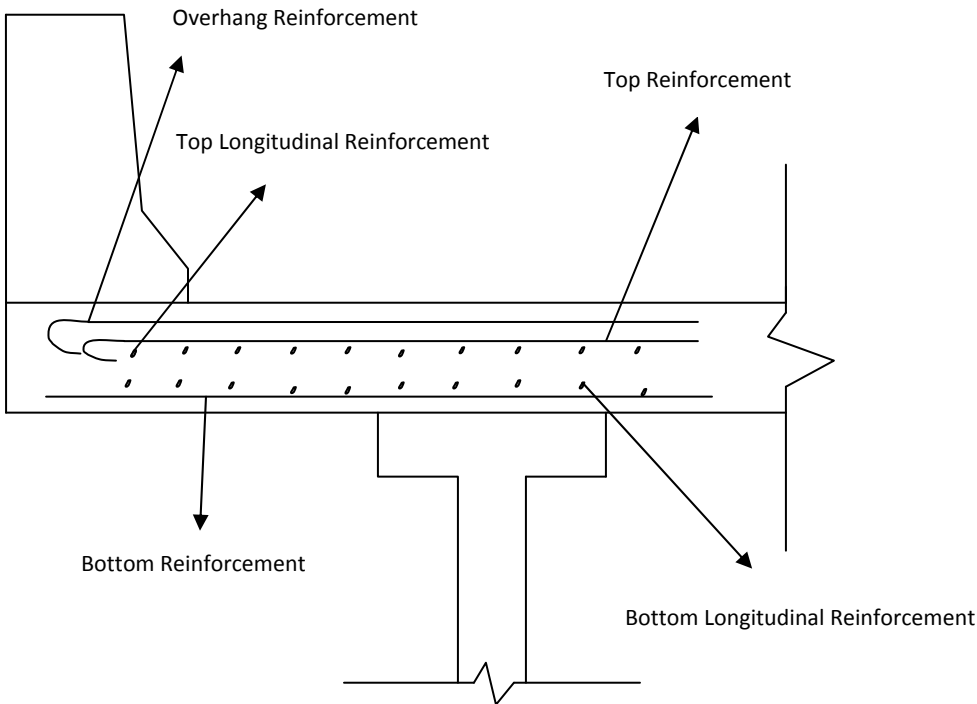


Figure 11: Composite deck illustration

As shown in Figure 11 above, there is top and bottom reinforcement which runs the width of the bridge. Both the top and bottom rebar are responsible for resisting bending in the lateral axis. The top and bottom longitudinal reinforcement runs the length of the bridge and resists bending in the longitudinal axis. Overall, all reinforcement is used to resist bending along different axis.

4.1.2 Composite Girder

Once all the loads were calculated, we calculated that the total factored moment was 1684 ft-k. This based on 6 foot spacing between each girder. In order to resist this moment, we used the

AISC Steel Construction Manual to select a W27X94 beam. This decision was based on the assumption of full composite action with the plastic neutral axis at the top of the steel beam.

Next we determined the number of shear studs. The $\frac{3}{4}$ " diameter studs we used were able to withstand a 17.2k force, so each beam would require a total of 162 to adequately resist the force. Then we checked the strength of the W section before the concrete hardened. Using the concrete and a construction live load of 20 psf, it was found that the selected steel section would not have sufficient moment capacity for use in un-shored construction. So we decided to try a W 27X102 with 175 shear studs. This had adequate moment resistance.

After checking the beam for moment capacity, we looked at deflection. First we made sure the service load deflection before the concrete hardened was within the limit of $L/360$. However, it was 9.3 inches, which is too high. But, we thought about what would happen if the center pier was left in as temporary shoring until the concrete hardened. If this happened, the deflection would only be .58 inches, well below the allowed 2.5 inches. Next we checked service live load deflection after the composite section was available and the design shear forces. Both of these were below the allowed values.

We determined that a W27X102 beam, with 175 shear studs, and at a spacing of 6 feet would be able to span the entire 88 foot distance between the two end abutments. Shoring will have to be provided, and this could be satisfied by leaving the center pier in until the concrete hardens. The W27X102 beam is six inches shallower than the current one and thus will increase the clearance.

After following the AASHTO guidelines we determined that this system would work. Using a composite design will complete two of Mass Highway's biggest goals: increasing visibility by removing the center pier and increasing the vertical clearance by reducing the bridge's profile.

4.1.3 Pre-stressed Concrete

Along with the design of a composite deck and steel girder system, we also investigated at a pre-stressed concrete system. The first step was to determine the cross section of the bridge, and this can be seen below in Figure 12. This figure shows the cross-section for the particular pre-stressed concrete system that we investigated. This cross-section also includes all major dimensions. Next we calculated the moment and shear loads. These can be seen in Table 5 and Table 6. These values were used in all the subsequent design calculations. All of the loads are the un-factored or service-level loads for designing of the girders. As can be seen in the table, the self-weight of the girders and the live loading (LL+IM) created the largest moment. This was expected due to the large amounts of concrete needed to create the bridge girders. However, with shear, the live loading had the largest effect, which corresponds to locating the design truck adjacent to an abutment.

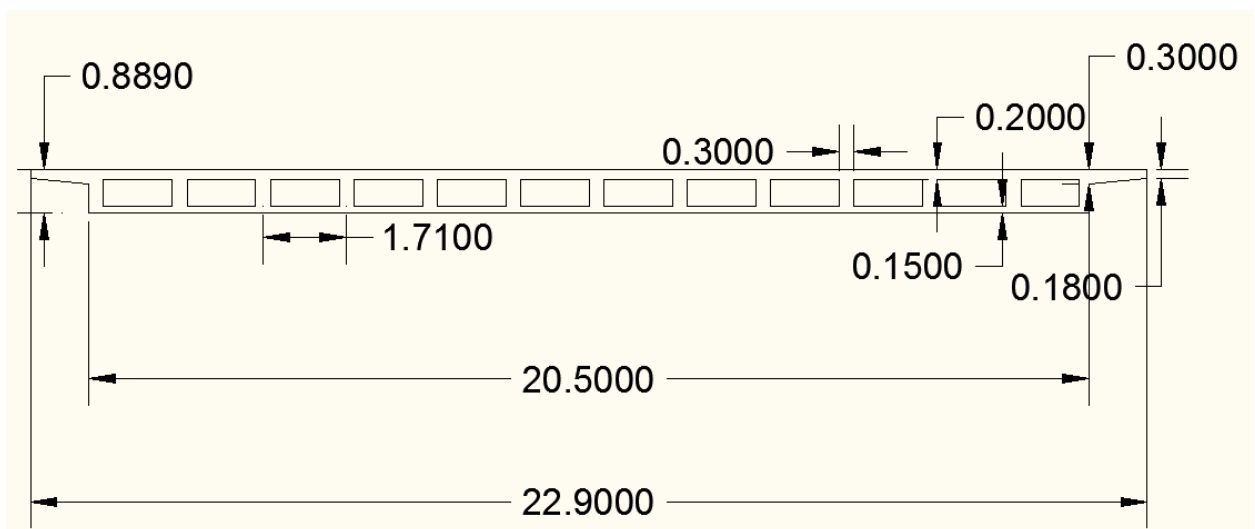


Figure 12: Cross Section of Pre-stressed Bridge

Table 5: Un-factored Moments due to Dead and Live Loads

	Un-factored Moments (KN-m)				
Distance (m)	Girder	Parapet	Wearing surface (interior)	Wearing surface (exterior)	LL+IM
0	-1226.99	-52.07	-168.79	-83.80	-1639.12
6.7 (1/4 L)	153.37	6.51	21.10	10.47	68.26
13.4 (1/2L)	613.50	26.04	84.39	41.90	1167.94
20.1 (3/4L)	153.37	6.51	21.10	10.47	361.51
26.8 (L)	-1226.99	-52.07	-168.79	-83.80	-2006.37

Table 6: Un-factored Shear

	Un-factored Shear (KN)				
Distance (m)	Girder (DC1)	Parapet (DC2)	Wearing surface (interior) (DW)	Wearing surface (exterior) (DW)	LL+IM
0	274.7	11.658	-37.788	18.76	455.97
6.7 (1/4 L)	137.35	5.829	-18.894	9.38	158.79
13.4 (1/2L)	0	0	0	0	-91.18
20.1 (3/4L)	-137.35	-5.829	18.894	-9.38	-152.28
26.8 (L)	-274.7	-11.658	37.788	-18.76	-213.38

Once loads were calculated, we looked at the losses in the pre-stressing cables. First we calculated the pre-stressed frictional losses, which can be seen in Table 5. In the table, E_p is the vertical distance between two points, L_p is the horizontal distance, and α is the sum of the change of angles. The total pre-stressed frictional loss is 22.9 Mpa. This data was used to create Table 8, pre-stress losses. In the table, Δf_{pF} is frictional loss, Δf_{pA} is anchorage set loss, Δf_{pES} is elastic shortening loss, Δf_{pTM} is time dependent losses, and Δf_{pT} is total losses. As can be seen in the table, the total losses are symmetrical around the center of the bridge. Table 9 shows the equations used to determine the pre-stress losses and the values for certain key variables.

Table 7: Pre-Stress Frictional Loss

Pre-stress Frictional loss							
Segment	Ep (mm)	Lp (m)	α (rad)	$\Sigma\alpha$ (rad)	ΣLb (m)	Point	Δf_{pF} (Mpa)
Near end	0	0	0	0	0	Near end	0
End to Mid-span	208	13.4	0.031045	0.031045	13.4	Mid-span	11.1
Mid-span to far end	208	13.4	0.031045	0.06209	26.8	Far end	22.9

Table 8: Total Pre-Stress Loss

Location (m)	Pre-stress losses							Force Coefficient	
	Δf_{pF} (Mpa)	Δf_{pA}			Δf_{pES} (Mpa)	Δf_{pTM} (Mpa)	Δf_{pT} (Mpa)	F_{pCI}	F_{pCF}
		Lpa	Δf	(Mpa)					
0.000	0.000	0.000	0.000	0.000	29.500	145.000	174.500	0.980	0.883
6.700	5.548	98.295	162.776	151.681	29.500	145.000	331.728	0.875	0.777
13.400	11.095	69.505	115.100	92.910	29.500	145.000	278.505	0.910	0.813
20.100	5.548	98.295	162.776	151.681	29.500	145.000	331.728	0.875	0.777
26.800	0.000	0.000	0.000	0.000	29.500	145.000	174.500	0.980	0.883

Table 9:Pre-stress loss Equations

Pre-Stress loss	Equation	Coefficient	value
Δf_{pF}	$f_{pi} (1 - e^{-(Kx + \mu\alpha)})$	k	6.6×10^{-7} mm
Δf_{pA}	$(1 - x/L_{pa})$	μ	0.25
Δf_{pES}	$(N-1)/(2N) E_p/E_{ci} f_{cpg}$	X	length
Δf_{pTM}	145 Mpa (Constant)	α	angle change in pre-stressing cable
		N	4=# of pre-stressing tendons
		E_p	197,000
		E_{ci}	24768
		f_{cpg}	Force at face of support

Next we looked at the moment coefficients. These are shown below in Table 10, where e is the distance from the cable to the center of gravity of a beam. Like the pre-stress losses, these too are symmetric about the centerline of the bridge. The moment coefficients were used to calculate the stresses in each section of the beam and the jacking force, shown in Table 11. In

this table, a negative sign represents tension and a positive sign represents compression. Once this table was complete, we calculated the amount of steel required. This was done by dividing the largest positive jacking force, 12892.29 kN, which we rounded up to 129,000kN, by the limit stress in the pre-stressing steel, 1448 MPa, which is a constant for our material. This gave us a result of 8670mm² required for pre-stressing steel.

Table 10: Moment Coefficients

Location (m)	Cable Path e (m)	Force Coefficients (m)		Moment Coefficients (m)				
		FpCI	FpCF	FpCIe	FpCFe	MsC	MpsCI	MpsCF
0.000	0.015	0.980	0.883	0.015	0.013	0.000	0.015	0.013
6.7	-0.167	0.875	0.777	-0.146	-0.130	0.085	-0.061	-0.045
13.4	-0.267	0.910	0.813	-0.243	-0.217	0.170	-0.073	-0.047
20.1	-0.167	0.875	0.777	-0.146	-0.130	0.085	-0.061	-0.045
26.8	0.015	0.980	0.883	0.015	0.013	0.000	0.015	0.013

Table 11: Jacking Force

Determination of Pre-stressing Jacking Force for an Interior Girder										
Top Fiber						Bottom Fiber				
Location	Stress (Mpa)				Jacking Force	Stress (Mpa)				Jacking Force
(x/L)	fDC1	fDC2	fDW	fLL+IM	Pj(kN)	fDC1	fDC2	fDW	fLL+IM	Pj(kN)
0	-6.48	-0.27	-0.89	-8.65	11288.92	5.64	0.24	0.78	7.54	-13850.09
6.7	0.81	0.03	0.11	0.36	-5166.95	-0.90	-0.04	-0.12	-0.40	178.13
13.4	3.24	0.14	0.45	6.16	-15766.83	-3.58	-0.15	-0.49	-6.82	10845.92
20.1	0.81	0.03	0.11	1.91	-7185.06	-0.90	-0.04	-0.12	-2.11	702.82
26.8	-6.48	-0.27	-0.89	-10.59	12892.29	5.64	0.24	0.78	9.23	-15238.99

After we calculated the jacking force and amount of pre-stressing steel required, we checked the concrete strength for an interior girder. The results of this can be seen in Table 12. In the table, the largest axial compression force is 70 Mpa. This value gets divided by .55 in order

to calculate the required f'_{ci} , which is 127MPa. Unfortunately, this value for f'_c was above the practical value for concrete which is 28 MPa (4 ksi).

Table 12: Concrete Stress

Concrete Stresses after Instantaneous Losses for the Interior Girder										
Location	Top Fiber Stress (Mpa)					Bottom Fiber Stress (Mpa)				
(m)	fDC 1	FpC1 * Pj /A	MpsC1 * Pj*Yt/I	fpsI	Total Initial Stress	fDC 1	FpC1 * Pj /A	MpsC1 * Pj*Yt/I	fpsI	Total Initial Stress
0.00	-6.48	16.31	10.00	26.32	19.84	5.64	16.31	-11.07	5.24	10.88
6.70	0.81	14.56	-41.54	-26.97	-26.16	-0.04	14.56	45.97	60.53	60.49
13.40	3.24	15.15	-49.70	-34.56	-31.32	-0.15	15.15	55.01	70.15	70.00
20.10	0.81	14.56	-41.54	-26.97	-26.16	-0.04	14.56	45.97	60.53	60.49
26.80	-6.08	16.31	10.00	26.32	20.24	5.64	16.31	-11.07	5.24	10.88

After following the AASHTO guidelines for a pre-stressed concrete bridge, we determined that it would not be possible to use pre-stressed concrete, given the span length and depth requirements. This was proven by the value needed for the strength of the concrete: it would have to have an initial strength of 127 Mpa, which is beyond practical applications.

4.1.4 Abutment Design

The removal of the center pier was necessary to fulfill Mass Highway's objective of increasing sight distance. However, in doing this it will increase the loading on each abutment. Typically, a center pier supports approximately 60% of the loading. With the removal of the center pier, the loadings on the abutments will more than double (Nabulsi, 2009). Due to a number of unknowns, such as soil profile and bearing strength of the existing abutments, an exact design for modifying the abutments was difficult.

After extensive research and conversations with a colleague who has experience with geotechnical engineering, our group came up with a possible solution (Perry, 2009). This

solution involves the use of micropiles to increase the bearing capacity of the existing abutments. Specifically, we decided on installing 3-5” diameter micropiles, to a depth of 20’ under each abutment. This depth was assumed based on conversations with our geotechnical resource (Perry, 2009). Our group was unable to find a soil profile of the immediate area which could have been used to calculate the exact depth of the micropiles. The installation process of these micropiles should take no longer than two weeks. The process for installing micropiles is as follows:

- Drill 20’ below existing abutment
- Place 5” diameter metal tube to designate a controlled volume
- Displace all soil from inside of metal tube
- Install pre-fabricated metal rebar inside of tube
- Pump concrete/grout inside tube and cap

Although this process will not require much time for completion, a traffic control plan for Rt.20 was still necessary. This traffic control plan is outlined in the next section.

4.2 Cost Analysis

In this section of the project, we looked at the initial and life-cycle costs. The implementation of sustainable techniques using the LEED guidelines to lower construction and maintenance costs was also investigated. The parts of the LEED guidelines we considered were materials and resources, which affected the initial cost; energy, which affected life-cycle cost; and sustainable sites, which affected both costs (U.S. Green Building Council, 2009). These cost analyses were necessary because for large, publicly funded projects, the bottom line is extremely necessary (includes both construction and maintenance costs). Also, it is important to know how much a project will cost throughout its life.

4.2.1 Initial Cost

Our group obtained unit prices from a number of different sources and different years. These unit prices were then adjusted based on inflation rates and location factors. Table 13 below outlines the unit prices for the construction materials, along with their source and year of record. In addition to material costs, the unit prices listed below also includes labor and equipment costs.

Table 13: Initial Cost Unit Prices

Material	Cost	Unit	Source	Year
Rolled W27x102 girder	\$193	LF	<i>R.S. Mean's Heavy Construction Index</i>	2009
4000 psi structural concrete	\$116	CY	<i>R.S. Mean's Heavy Construction Index</i>	2009
Type F parapets	\$95	LF	Oklahoma DOT Website	2009
Wearing Surface	\$3.60	SY/in	Maryland DOT Website	2003
AASHTO Type II Pre-stressed Girder	\$163	LF	Florida DOT Website	2005
AASHTO Type III Pre-stressed Girder	\$175	LF	Florida DOT Website	2005
AASHTO Type IV Pre-stressed Girder	\$190	LF	Florida DOT Website	2005
AASHTO Type V Pre-stressed Girder	\$225	LF	Florida DOT Website	2005
AASHTO Type VI Pre-stressed Girder	\$250	LF	Florida DOT Website	2005
Galvanized Reinforcing Steel	\$1,150	Ton	<i>R.S. Mean's Heavy Construction Index</i>	2009
Bridge Bearings	\$1,750	Ea	Steve Bowmen (SEP Bearings)	2009
Drilled Micro Piles (Metal Pipe)	\$98	LF	<i>R.S. Mean's Heavy Construction Index</i>	2009

As illustrated above in Table 13, the unit price for the wearing surface was from 2003 and the unit prices for the pre-stressed girders were from 2005. In order to validate these results for the current year, we included inflation rates for the past years leading up to 2009. The

inflation rates were obtained from *R.S. Mean's Heavy Construction Index* and are shown below in Table 14.

Table 14: Inflation Rates

Year	Average Inflation Rate
2008	3.85%
2007	2.85%
2006	3.24%
2005	3.39%
2004	2.68%
2003	2.27%

These inflation rates, along with the location factors retrieved from the *R.S. Mean's Index* were used to adjust the unit prices listed in Table 13. Our final adjusted unit prices are outlined in Table 15.

Table 15: Unit Prices Adjusted for Inflation and Location

Material	Cost	Unit	Source	Year
Rolled W27x102 girder	\$193	LF	<i>R.S. Mean's Heavy Construction Index</i>	2009
4000 psi structural concrete	\$116	CY	<i>R.S. Mean's Heavy Construction Index</i>	2009
Type F parapets	\$95	LF	Oklahoma DOT Website	2009
Wearing Surface	\$3.71	SY/in	Maryland DOT Website	2003
AASHTO Type II Pre-stressed Girder	\$168	LF	Florida DOT Website	2005
AASHTO Type III Pre-stressed Girder	\$181	LF	Florida DOT Website	2005
AASHTO Type IV Pre-stressed Girder	\$196	LF	Florida DOT Website	2005
AASHTO Type V Pre-stressed Girder	\$233	LF	Florida DOT Website	2005
AASHTO Type VI Pre-stressed Girder	\$258	LF	Florida DOT Website	2005
Galvanized Reinforcing Steel	\$1,150	Ton	<i>R.S. Mean's Heavy Construction Index</i>	2009
Bridge Bearings	\$1,750	Ea	Steve Bowmen (SEP Bearings)	2009
Drilled Micropiles (Metal Pipe)	\$98	LF	<i>R.S. Mean's Heavy Construction Index</i>	2009

Finally, we multiplied these unit prices by their quantities defined within our design to obtain a total initial cost. All quantities, other than the micropiles, were directly based off of the design. The number of reinforcing bars used in calculating the quantities is shown in Table 16. Table 17 below shows the dimensions of the bridge which were also used in calculating the quantities used. Figure 13 below illustrates the top view of the Rt. 122 overpass. Our final results, including the material list, unit prices, quantities and total initial cost are outlined below in Table 18. Overall, we came up with a total initial cost of \$340,617.

Table 16: Spacing of Reinforcing Steel Used to Calculate Quantity of Galvanized Reinforcing Steel

Positive Moment	# 5 bars @ 7"
Negative Moment	# 5 bars bundled with #3 bars @ 4"
Bottom Longitudinal	# 5 bars @ 7"
Top Longitudinal	# 4 bars @ 10"
Overhang	# 5 bars bundled with #3 bars @ 4"

Table 17: Bridge Dimensions Used to Calculate Various Quantities

Bridge Dimensions	unit	Source
87.93	ft.	Mass Highway
75.13	ft.	Mass Highway
8	in.	AASHTO Specifications
1	in.	AASHTO Specifications
13	ea.	Composite System Design

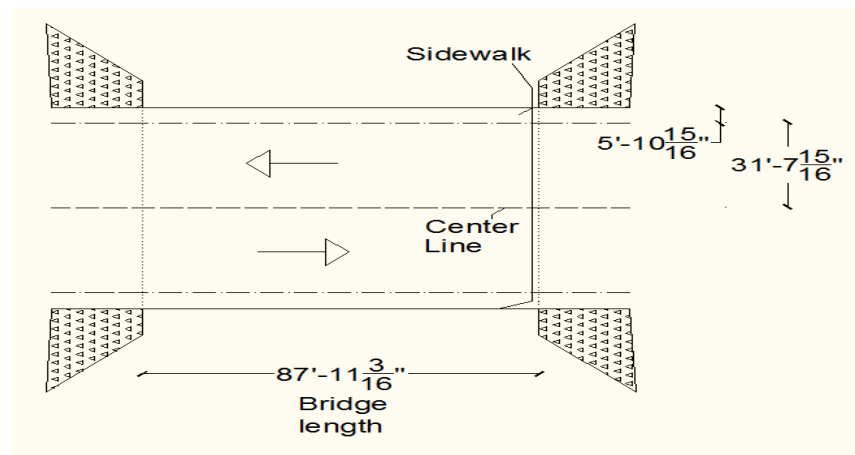


Figure 13: Top View of Rt. 122 Overpass

Table 18: Total Initial Cost of Construction Materials

Material	Cost	Unit	Quantity	Total Cost
Rolled W27x102 girder	\$193	LF	1,144	\$220,792
4000 psi structural concrete	\$116	CY	167.9	\$19,476
Type F parapets	\$95	LF	176	\$16,720
Wearing Surface	\$3.71	SY/in	753	\$2,794
Galvanized Reinforcing Steel	\$1,150	Ton	20.5	\$23,575
Bridge Bearings	\$1,750	Ea	26	\$45,500
Drilled Micropiles	\$98	LF	120	\$11,760
Total				\$340,617

Along with the cost of construction, the cost of demolition was also considered. This was figured at \$50/ft². For the Rt. 122 overpass, which is about 6,600 square feet, the cost of demolition would be about \$330,000 (Florida DOT, 2009). We recommend, and assume, that the contractor will recycle construction debris because it will allow them to receive between \$130 per ton of steel instead of paying \$70 a ton to throw it away (Bartner, 2009). Steel is a very recyclable material and almost all steel has some amount of recycled material in it.

Concrete is just starting to get recycled more. It returns a lot less than steel because it has to be separated. However, the separating cost of about \$10 a ton equals the return cost (Bartner, 2009). Therefore the contractor does not need to pay the disposal fee. Also, once separated, the steel rebar can be sold off for recycling. The separated concrete, in turn, can be sold to an aggregate company and used to make more concrete. This concrete is highly sought after because it has a higher strength-to-weight ratio than normal concrete (Construction Materials Recycling Association, 2009).

For this project, Mass Highway estimated a total cost of approximately \$4.5 million. This is far above our estimation of approximately \$670,000. Their estimation was much larger

because we mainly accounted for the costs that would differ between possible building materials. The initial intent of this section was to use it as a comparison to determine the most economically feasible design, if more than one building material would have fit the project's limitations. Some factors that were left out of this estimation, and would be similar in any design were: cost of traffic management, design costs, overhead and profit, road painting costs, sidewalk costs, bridge rail costs, storage space rental fees, transportation costs, and the cost to resurface Rt. 20 after demolition.

4.2.2 Life-Cycle

First, we found out that this bridge will be expected to last 75 years (Nabulsi, 2009). Next, we looked at bridge maintenance. This was complicated by the fact that Mass Highway has no set schedule for bridge repairs. Instead, every other year, Mass Highway has an inspector review the bridge to determine its quality (barring no serious incident in which case this will happen more frequently) (Nabulsi, 2009). This inspector looks at every part of the bridge and rates it, then based on this rating, action might be taken to repair parts of the bridge. Because of this practice, it was difficult to get a standard repair list. Every bridge is different, and some bridges might need to be resurfaced in 8-year intervals while others in 10 years.

Also, a standard repair cost was difficult to establish because much of the repairs are spot repairs. This is especially true for the reinforced concrete decking. With reinforced decking the cost of repair may be estimated by square foot of damaged concrete or linear foot of cracking (Idaho Transportation Department, 2009). Complicating this even more is that sometimes there are repairs, which can sometimes be very costly, that are not normal. These would include things

like a catastrophic collision, severe erosion, and errors in construction causing excess maintenance.

In our life-cycle cost analysis, we decided to focus on three main areas: the wearing surface, the decking, and the superstructure. These three areas are generally where the most repairs take place and are the most constant from bridge to bridge. Typical repairs, unit costs, and their frequency of application are outlined in Table 19 below. We did not look at elastomeric pad repairs because modern pads can last the whole life of the bridge.

Table 19: Life-cycle repair frequencies

Repair	Area ft ²	Unit Price (\$)	Cost (\$)	Frequency (Years)	Source
Non- Destructive Asphalt Testing	6600	0.2	1320	4	(Carmichael, 2009)
Repave/milling	6600	2	13200	15	(Paving, 2009)
Wearing Surface Sealing	6600	0.2	1320	4	(Paving, 2009)
Bridge Inspection	-	-	800	2	(Nabulsi, 2009)
Steel Painting	2002	34	68000	15	(Zayed, 2002)
Concrete Patch and Repair	330	100	33000	15	(Idaho Transportation Department, 2009)

As can be seen above, the steel painting is the most expensive part of bridge maintenance in terms of total cost. This was the only maintenance procedure accounted for the superstructure because it is the only repair that will occur at a specific frequency. All other repairs are done on an inspection basis, and their frequency can vary greatly.

When calculating the area for the painting and concrete repair, we used a percentage of the actual area. There is 8008 ft² of paintable steel (7 feet²/foot of girder *88 feet/ girder*13 girders).

However, this is a spot repair and normally only 25% of the bridge will have to be repainted (Zayed, 2002). The estimated repair cost is for a 3-coat zinc paint process and includes the cost of traffic disruption. With concrete, the same idea of spot repairs is true. But, this is usually less than 5% of the surface area of concrete will have to be repaired (Idaho Transportation Department, 2009). The estimated repair cost includes fixing cracking and spalling.

We decided not to include the salvage value of the bridge at the end of its service life because it is money that will not be collected for 75 years. In 75 years recycling unit prices will change due supply and demand. Supply and demand would make them change at a rate different than a standard inflation rate. So it would be difficult to produce a reasonably accurate estimate of the return from the salvage of the bridge.

For sustainable practices that wouldn't have a significant impact on the initial cost of our project, but would reduce costs in the long term, we looked at lighting and water control. We looked into the lighting and found that with a bridge of this width, lighting would not be necessary (Attlesey, 2001). However, water control is something that could be applied to our project. This, along with increased vertical clearance and sight distance, was one of the original issues presented by Mass Highway. After visiting the site and having conversations with a designer at Mass Highway, our group came up with a simple solution to the drainage issue.

Figure 14 and Figure 15 below illustrate the location of the drains underneath the Rt. 122 overpass. As shown below, there are drains present on Rt. 20 Eastbound and Rt. 20 Westbound. The pooling of water under the bridge is not caused by lack of drainage structures, but rather by the grading of the road. Currently the road's wearing surface slopes away from the drainage structures. Re-grading Rt. 20 would force the water towards the drainage structures and would eliminate the pooling problem. After construction of the overpass is completed, Rt. 20 should be

re-graded in order to solve the pooling problem. Re-grading Rt. 20 would cause all water to gather in the catch basins, rather than pooling under the overpass.



Figure 14: Drainage Structures on Rt. 20 Eastbound Under Rt. 122 Overpass



Figure 15: Drainage Structures on Rt. 20 Westbound Under Rt. 122 Overpass

After we outlined the elements of the life-cycle cost, we figured out how much the repairs would cost for the life of the bridge, this is summarized in Table 2020 below, in Appendix E there is a breakdown of the repairs. In the table we calculated the cost of repairs for the life of the bridge. We calculated the present worth of the life-cycle cost using a 6% discount rate and a 3% inflation rate. Improvements in drainage were not calculated into this table because it was an initial construction cost. However, it could lessen the life-cycle cost of the bridge because this improvement will be something that makes it safer for drivers and ultimately lessen the chance of a collision into the abutments. In addition, it could reduce the possibility of erosion and undermining of the abutments.

Table 20: Life-cycle Repairs

Year interval	Repair	Total Accrued Costs Based on Initial Cost + 3% Inflation Rate	Present Worth
15	Repave/ Milling	\$1,559,800.00	\$185,000.00
	Steel Painting		
	Concrete Repair		
4	Asphalt testing	\$175,100.00	\$20,100.00
	Asphalt Sealing		
2	Bridge inspection	\$110,000.00	\$12,650.00
	Total	\$1,844,900.00	\$217,750.00

4.2.3 Cost Summary

We looked at both the initial cost and life-cycle cost of this bridge. These two items can be compared to each other in order to show the complete cost of this project, by using present worth from the life-cycle analysis. The initial and life-cycle cost are shown below in Table 21. As can be seen in the table, the total present worth of the bridge is approximately \$887,750. The life-cycle cost analysis shows that maintenance costs about one fourth of the life-cycle cost of this bridge.

Table 21: Initial and Life-Cycle Cost

	Cost
Initial	\$670,000
Life-Cycle	\$217,750
Total	\$887,750

4.3 Construction Phase

The construction phase included devising a traffic control plan, selecting the material for the primary members, and laying out the site for storage of these materials and access for operating the heavy machinery used in the construction of the new overpass. The traffic control plan is split into two sections. The first step was to explore a number of traffic management solutions for both Rt. 122 and Rt. 20. These solutions included detours, lane closures, the use of a temporary bridge, and protective structures to improve safety for traffic on Rt. 20. Material selection for the construction phase would help to choose between different materials for their ease in construction. Then, we mapped out the site and identified areas that would be suitable for storage.

4.3.1 Traffic Control

From the start of this project, we knew that one of the largest issues in the construction planning for this bridge was going to be traffic control. This is because both roads are very busy. Rt. 20 is a major industrial road with heavy traffic. Together, with Rt. 122, they are pivotal in linking Worcester to the Mass Pike and surrounding areas. In addition to this, where these two roads cross is an extremely urbanized area making traffic and pedestrian control much more difficult.

4.3.1.1 Route 122

The first part of traffic control we addressed was a plan for Rt. 122. This is a very busy road with daily traffic of 24,400 vehicles and 15% of which are trucks (Massachusetts Highway Department , 2009). After talking to a project designer, Mr. Nabulsi, at Mass Highway, we found that there were three possible options for traffic control (Nabulsi, 2009): re-routing traffic completely around the bridge, only closing part of the bridge at a time and allowing limited traffic, or installing a temporary bridge.

4.3.1.2 Detour

The first option considered was a detour. A detour would allow for the entire bridge to be closed down, and this would help the project in many ways. First, it would allow complete separation between the construction site and outside interferences. This is much safer for everyone involved. It would also allow contractors to have more space for operations, which would shorten the total construction time (Washington State Department of Transportation, 2006).

However, a complete closing of the bridge would be difficult because it has a very high traffic volume. Closing it would mean re-routing traffic in an already congested area, with the shortest detour being 2km, shown in Figure 166, as stated by Mass Highway (Massachusetts Highway Department , 2009). This bridge carries more than local traffic, and when trying to detour such users, confusion can occur and it should be avoided (Washington State Department of Transportation, 2006). Because of this fact, using a complete detour would not be the best solution if there is a strategy for the bridge to remain at least partially open.

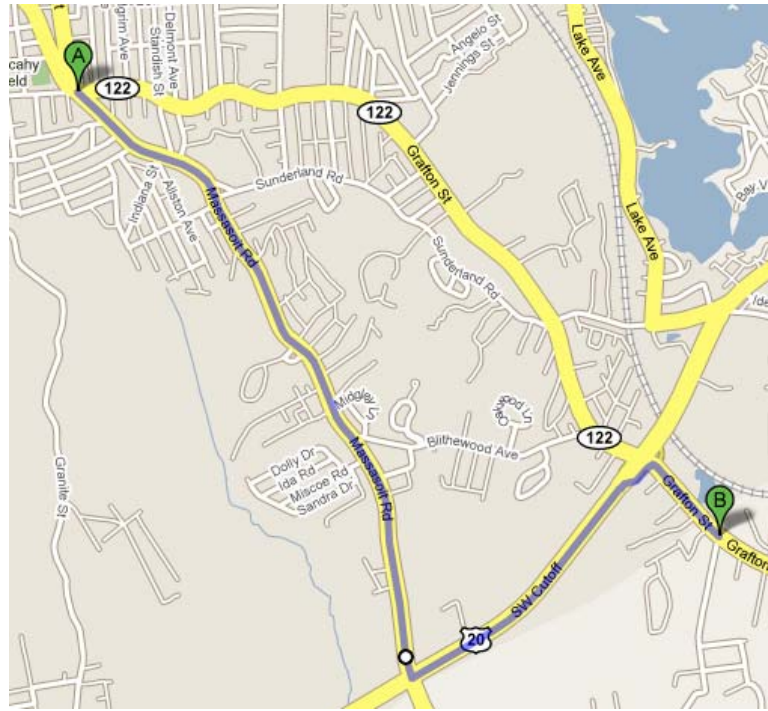


Figure 16: Detour for southbound traffic on Rt 122 (taken from Google Earth)

4.3.1.3 Partial Closing

Next, we considered a partial closing. The benefit of a partial closing is that no detour is necessary. Traffic can continue to travel the same route, which eliminates a lot of confusion and hassle for drivers. However, there are also some adverse effects. First, traffic would be slowed down; travel delays would depend on the number of closed lanes and the time of day. Second, there would be limited work space so construction would proceed slower than if the bridge was completely closed. Last, because traffic would be moving close to the construction, there would be an increased safety risk for the workers (Massachusetts Highway Department, 2009). Along with increased risk to drivers and construction works, pedestrians and bicyclists also would be closer to the construction, adding more congestion and safety risks to the project site. Construction operations would also have to be coordinated with the lane closures.

Despite these issues, a partial closure would allow traffic to continue, which was a serious downfall in the complete bridge closure. Also, looking at the current bridge design, it will be possible to do a partial lane closure because the bridge is over 60 feet wide from curb to curb. This could allow 2 lanes of traffic (each 11 feet wide) to fit on one side while the other side is closed.

4.3.1.4 Temporary Bridge

Last, we considered the erection of a temporary bridge. A temporary bridge is effective for long-term building projects when the bridge has to be closed. It offers all the benefits of a detoured bridge, such as increased work space and a high level of project safety, yet it would not require traffic to be detoured.

However, the procurement and erection of a temporary bridge is expensive. It requires sufficient area near the current bridge to install it. There has to be ample room to install abutments and adjust alignments for traffic flow. Also, time needs to be added to the project in order to install and remove the temporary bridge.

After looking at the site, we determined that a temporary bridge would not work. The area where Rt. 122 crosses over Rt. 20 is very built up, and there are on/off ramps extremely close to the bridge. The figure below illustrates this fact. It shows ramps and businesses that are located close to the bridge. These facts would make it practically impossible to install a temporary bridge on either side of the existing bridge.



Figure 17: Areal view of Rt. 122 showing obstacles for a temporary bridge (Google Earth)

4.3.1.5 Rt. 122 Recommendation

After looking at the three options, we decided that the best option was a partial closing. A temporary bridge would be impossible given the limited space. Also, when considering the objectives of a Traffic Management Plan, a partial closing would allow traffic to continue on the same route while having minimal delays or detours. In addition to a maintained traffic flow, it also filled the rest of our goals. In a partial closing, the workers, motorists, and everyone else who comes in contact with the project site can still be very safe if the proper protocol is followed. The only real drawback we determined was that in a partial closing, construction time can take longer due to space constraints.

The first option we investigated with a partial bridge closing was to close half of the bridge at a time. The bridge is 75' 1 1/2" out to out, then once the sidewalks are subtracted out of this, it is 63' 3 7/8" (curb to curb), so this would allow 31' 7 15/16" for road traffic on each side (Massachusetts Highway Department , 2009) as shown in Figure 188.

Once we decided where to initially divide the bridge, we had to confirm that the existing girder layout would provide sufficient support to support portions of the bridge width. As shown below in Figure 19, there are 13 girders spaced 6' on center. So, if the bridge is divided over the center girder, there will be support for the outside edge.

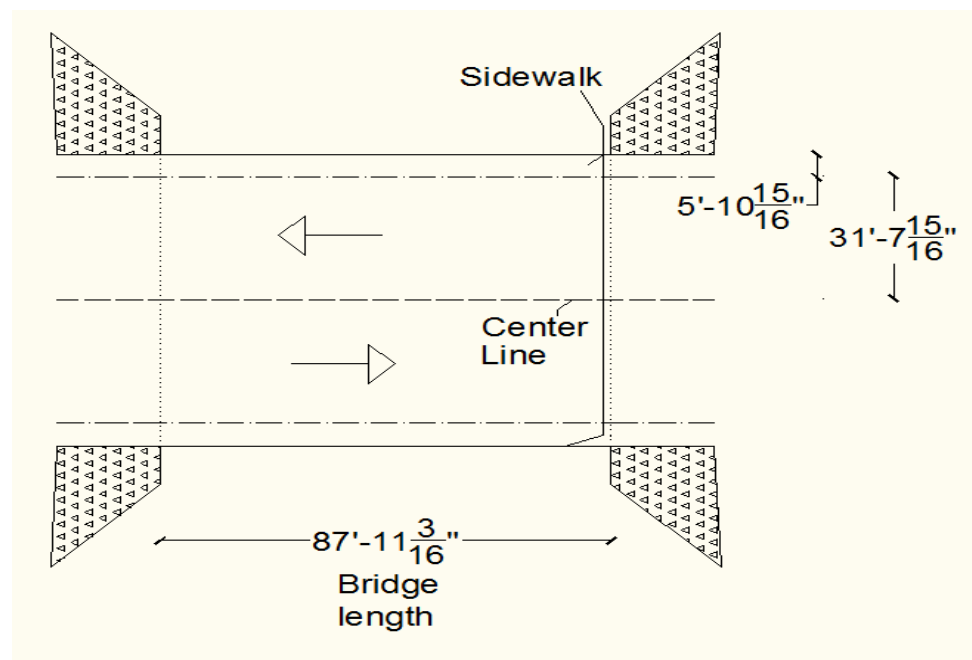


Figure 18: Dimensions of bridge showing sidewalks and center line

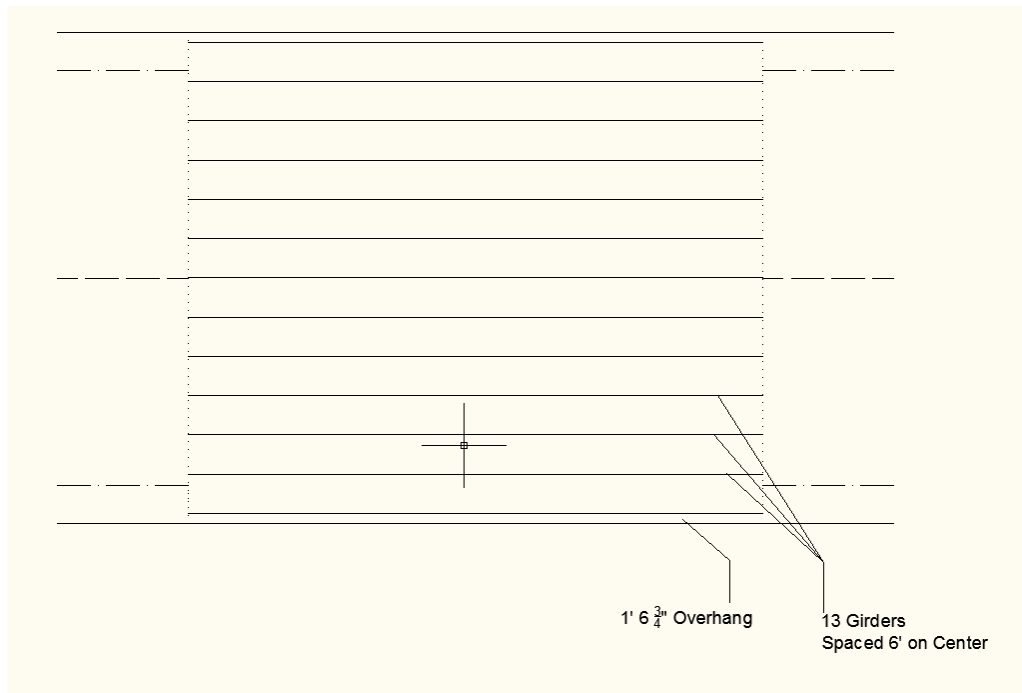


Figure 19: Girder layout

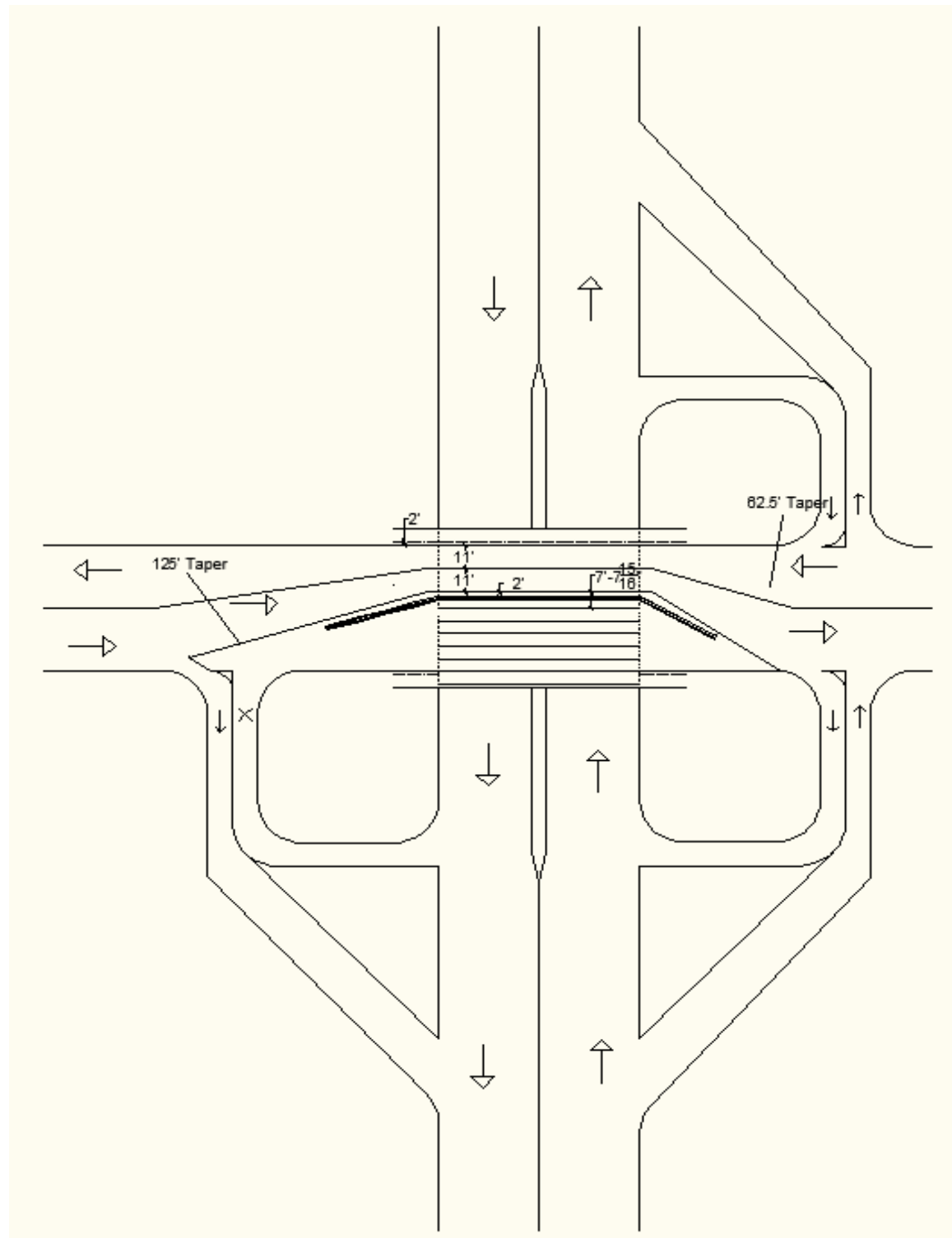


Figure 20: The layout of traffic flow during construction with the southbound lane of Rt. 122 closed

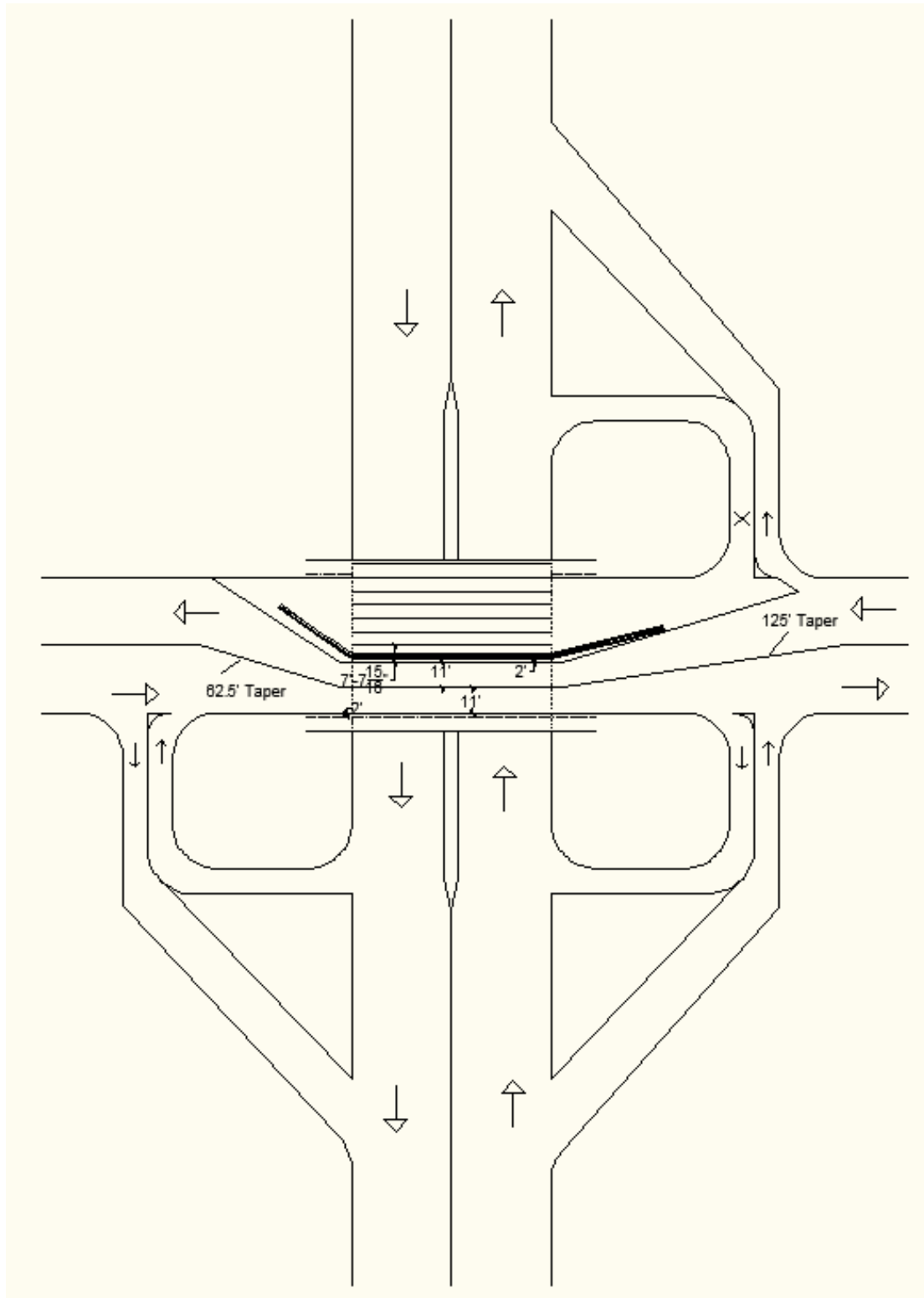


Figure 21: The layout of traffic flow during construction with the northbound lane closed

Next, we had to determine a traffic layout; this is shown above in Figure 2020 and Figure 2121. These diagrams show how partially closing the Rt. 122 overpass affects the immediate area. This includes lane shifts and ramp closures. In creating our traffic layout, the most important factor was safety. A partial closing has much more interaction between all parties in the construction site than a complete closure does, so this heavily affected our strategy. One of the influential aspects of safety is the space provided to shift lanes (U.S. Department of Transportation, 2007). Providing adequate time and warning of any alterations to traffic patterns can help greatly reduce the risk of an accident happening. In order to do this, we calculated the proper taper distance from the chart and tables below in Figure 22 and Figure 233.

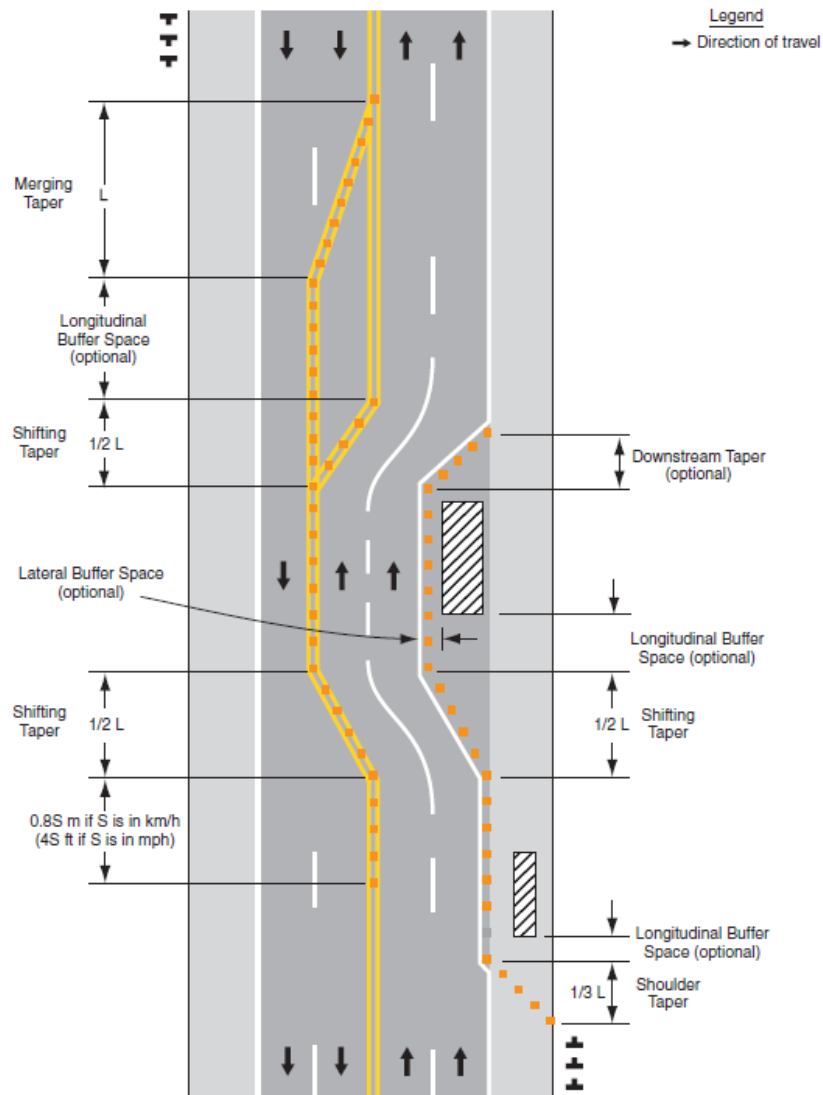


Figure 22: Standard traffic patterns (U.S. Department of Transportation, 2007)

Type of Taper	Taper Length (L)*
Merging Taper	at least L
Shifting Taper	at least 0.5L
Shoulder Taper	at least 0.33L
One-Lane, Two-Way Traffic Taper	30 m (100 ft) maximum
Downstream Taper	30 m (100 ft) per lane

Speed Limit (S)	Taper Length (L) Meters	Speed Limit (S)	Taper Length (L) Feet
60 km/h or less	$L = \frac{WS^2}{155}$	40 mph or less	$L = \frac{WS^2}{60}$
70 km/h or more	$L = \frac{WS}{1.6}$	45 mph or more	$L = WS$

Where: L = taper length in meters (feet)

W = width of offset in meters (feet)

S = posted speed limit, or off-peak 85th-percentile speed prior to work starting, or the anticipated operating speed in km/h (mph)

Figure 23: Formulas for Tapers (U.S. Department of Transportation, 2007)

In calculating the taper distances for the proposed lane shifts, we decided to reduce the speed limit to 20 mph. We did this for two reasons. First, it allowed the taper distance to be significantly shorter. A longer taper would have an adverse effect on traffic because of the nearby highway ramps and roads running perpendicular to Rt. 122. Having a longer taper would cause more of these roads and ramps to be closed during construction. Second, reducing the speed allows for more time for drivers to become aware of the construction site, thus reducing accidents (Washington State Department of Transportation, 2006).

Along with the distance to a lane shift, lane and shoulder widths greatly affect user safety and were an essential part of this project. For construction, the lanes will be 11' wide in each direction (American Association of State Highway and Transportation Officials, 2004). Also,

there will be a 2' shoulder on either side. However, it is acceptable in construction projects to have a temporary shoulder which is smaller than the standard of 10', especially if it is only for a limited distance (U.S. Department of Transportation, 2007). This is because a shoulder has two main purposes: allowing for pedestrian traffic in the event there is no sidewalk, and providing emergency space for motor vehicle traffic (U.S. Department of Transportation, 2007). In the case of this bridge, there would be a sidewalk which would provide safety for pedestrian traffic. In addition to this, the shoulder would only be 2' wide for 88 feet, and then it would open up again and allow space in the event of a motor vehicle emergency.

Despite the best efforts to avoid any detours, one will be necessary during the construction of each side of the new bridge. During construction on the southbound side of Rt. 122, the westbound off ramp of Rt.20 will be closed. Then, during construction on the northbound side of Rt. 122, the eastbound off ramp of Rt. 20 will be closed.

As can be seen in Figure 2020 and Figure 2121, the off ramps are located well within the tapers. Having a merging lane well within a taper and very close to the construction site is unsafe for motorists, workers, pedestrians, and anyone else who would be involved. This will prevent traffic from going off of Rt.20 and onto Rt. 122. Although detours were what we were trying to avoid by creating a partial closing, these required detours will only have a small affect on traffic.

There are two methods which will be applied to help in this situation, they are warning traffic about the closings and offering a possible detour (Washington State Department of Transportation, 2006). Below are the detours which will be necessary. Figure 244 shows the labeled detour that can be used to compensate for the ramp closing during construction on the southbound side of Rt.122.

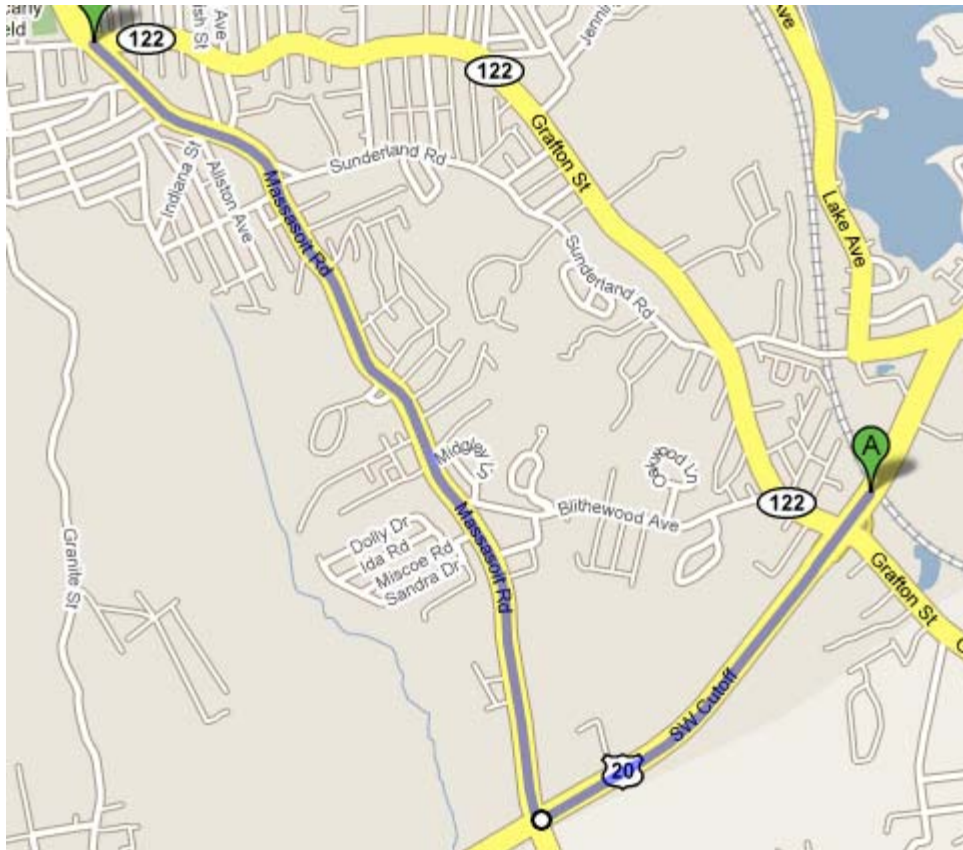


Figure 24: Shows a detour to get around ramp closings (Google Maps)

This detour is the same one that was considered when we initially thought about completely closing the bridge. However, in the case of the partial closing, the expected traffic volume will be much less than for the bridge closure. The only traffic being diverted is travel from Rt.20 East to Rt.122 South.

During construction on the northbound side of Rt. 122, traffic impact on Rt. 20 will be reduced. Traffic will be able to take normal routes to go into downtown Worcester, which is up Massasoit Rd. The only limitation is when the destination is south of the intersection of Rt. 122 and Massasoit Rd. In this instance, the normal traffic pattern is to travel up Rt. 122; however, during construction, traffic will be detoured up Massasoit Rd, and then down Rt. 122, Figure 255 shows this.

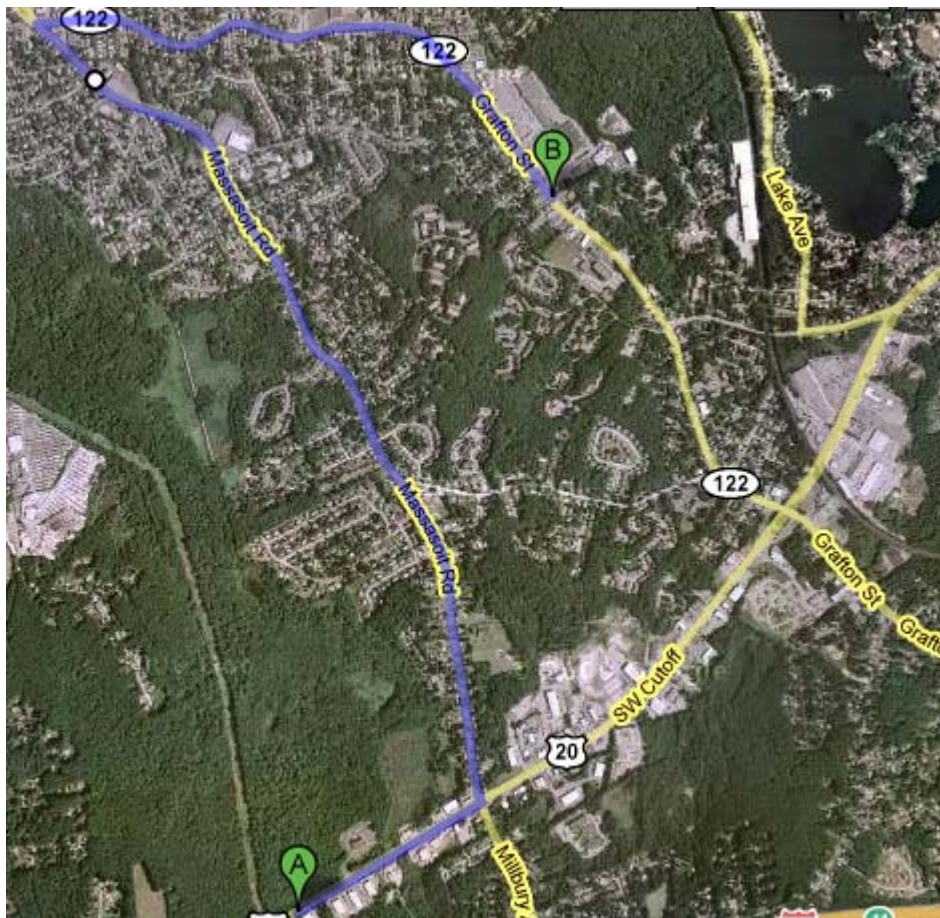


Figure 25: Detour of Eastbound off ramp of Rt. 20

4.3.2 Rt. 20

The effect that the bridge construction will have on Rt. 20 will be far less than the effect it has on Rt.122. There were two solutions that were taken into consideration when dealing with the traffic control plan of Rt. 20. The first solution was to close down either Rt. 20 eastbound or Rt. 20 westbound. Due to the fact that there are two travel lanes in each direction, traffic could still move in both directions if travel is reduced to one lane. With the technique for traffic control on Rt. 122 (close half of the bridge at a time), the bridge would have to be disassembled in quarters as shown. Figure 26 and Figure 27 below show the traffic plan that would be necessary when the northbound lane of Rt.122 is being removed. The red squares represent the portions of

the bridge which are being dismantled. These figures show the implications to Rt. 20 when two different quarters of the bridge are being dismantled. The same traffic pattern would occur during the removal of the sound bound lane. During the construction phase, there will be no impact on Rt. 20 because the steel girders will be erected in their entirety. However, at the time between when the existing steel girders are being removed and the new ones are being placed, there will be no lagging to protect motorists passing below on Rt. 20 from falling objects. During this time the only phase of construction which will be taking place is the removal and installation of girders. At this time, the decking will be completely removed and there should be no risk of falling objects. These figures represent the steps that must be taken before construction is able to begin.

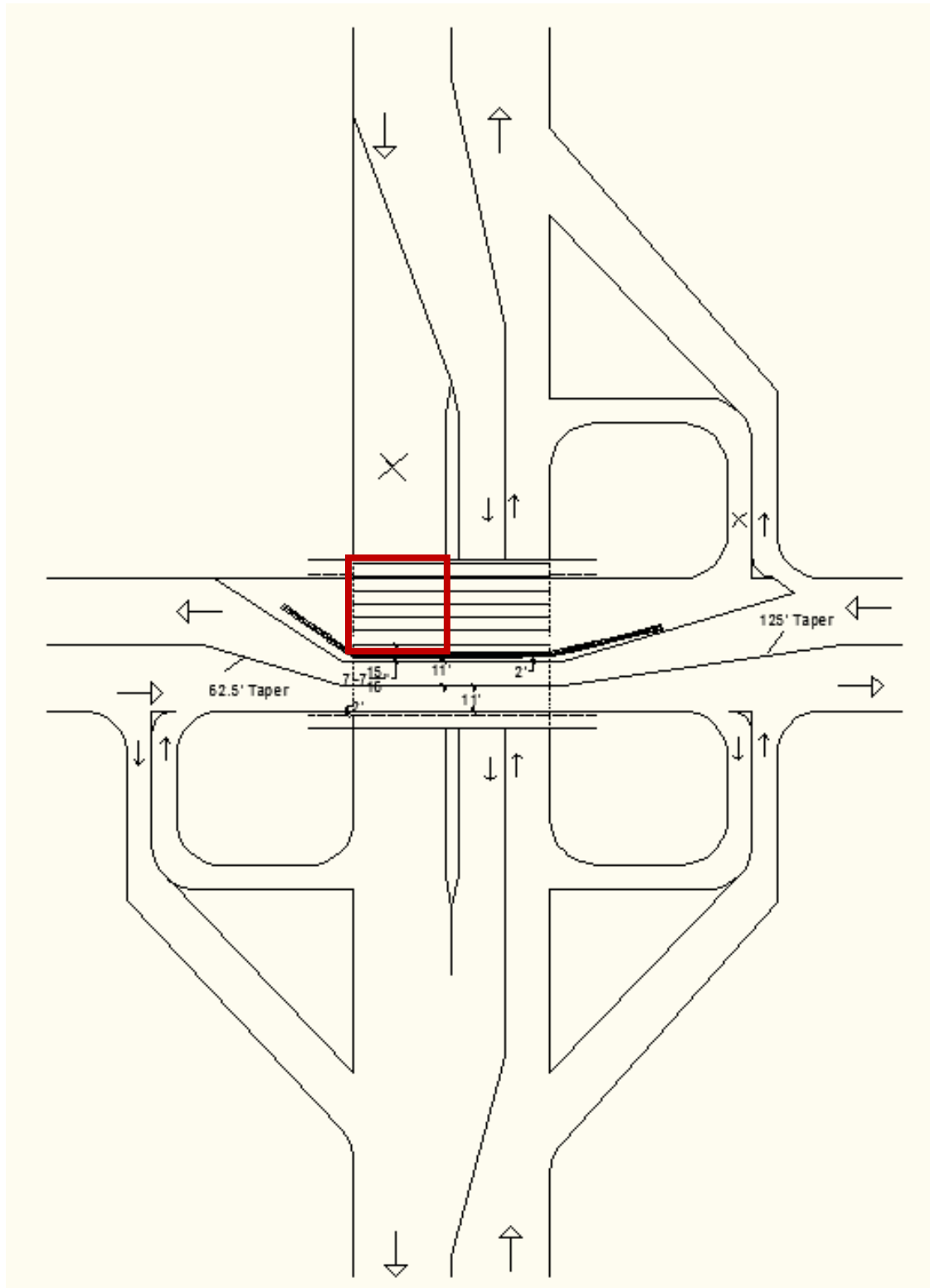


Figure 26: Traffic Control Plan for Rt. 20 When the Eastbound Lane is Closed

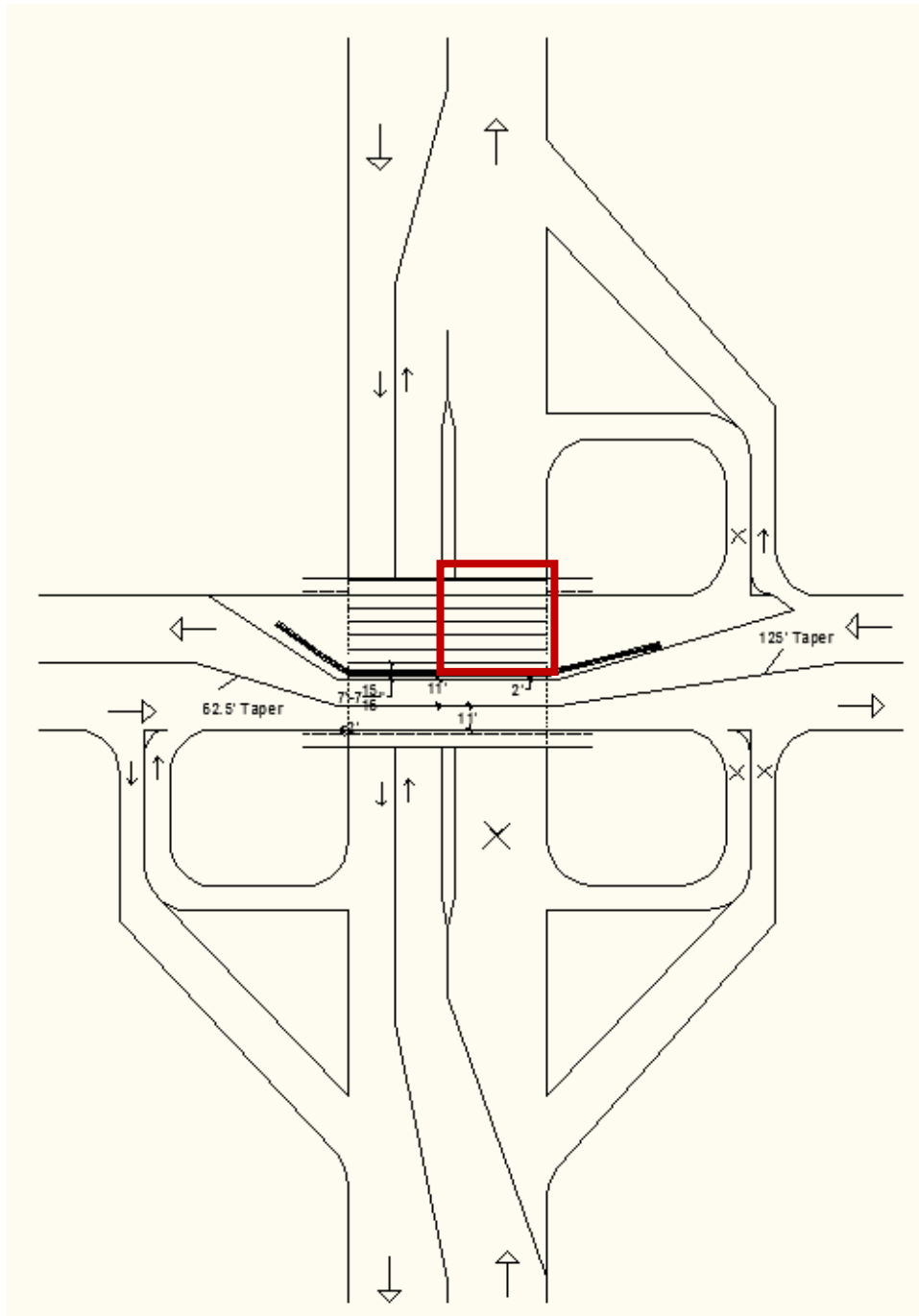


Figure 27: Traffic Control Plan for Rt. 20 When the Westbound Lane is Closed

Originally, our group saw this technique as being the most inexpensive option, but it also had its flaws. First, there would be a minimum of three lane shifts during the demolition process.

Rt. 20 is a highly traveled industrial road, and these lane shifts would most likely lead to more congestion on the roadway. Also, if this technique was used, two-lane traffic would be forced to travel in one lane. The original speed limit of 50 mph would have to be lowered significantly and traffic would run less efficiently.

The second technique does not involve closing down any roadways. The main objective of this alternate traffic plan is to protect traffic from falling debris during the demolition stage. The second technique utilizes tongue and groove 3"x8" lagging boards that are placed on the bottom flange of the existing I-Beams. These boards protect the traffic traveling under the bridge from falling debris. A crew would put these boards on the flanges using a man lift. They would also install a layer of poly in order to seal the joints. The poly would keep any of the smaller particles/dust from disturbing traffic below. After the removal of the concrete deck, the lagging boards and poly will be removed along with the existing girders. Then, the new girders will be erected. Before construction of the new concrete deck, the lagging boards and poly will be re-installed to protect traffic on Rt. 20.

Our group decided that this technique should be used. Using the lagging boards to protect traffic below the bridge would not require any road closures or lane shifts for extended periods of time. In order to affect traffic less, the process of installing the lagging boards and poly could be done at night. During the dismantling and installation of the new girders, there will be no protection. This will not be an issue because the deck would have already been removed and the time of erecting the girders would be short. As previously mentioned, Rt. 20 is a highly traveled industrial road. Our main objective when designing the traffic plan for Rt. 20 was not to disturb traffic. This technique would allow traffic to flow with little interference. Figure 28 below

illustrates this technique being used on another bridge in Worcester (on I-290 above Lincoln Street).



Figure 28: Lagging Being Used as a Protective Barrier Below I-290

4.3.3 Material Selection

For material selection, the principal objectives were to increase vertical clearance and improve sight distance as desired by Mass Highway. Our investigation of structural steel and pre-stressed concrete demonstrated that the steel system could satisfy both objectives.

4.3.3.1 Site Design

For any construction project of this magnitude, site design is vital for storage of both materials and machinery. Through our numerous visits to the site, along with the use of Google Earth, we were able to designate areas with sufficient space for storage. The map below outlines these areas. As one can see in Figure 29, there are a number of different locations near the existing overpass. The bulk of the storage will be for the heavy machinery due to the fact that the majority of the pre-fabricated beams can be installed directly off the flatbeds on which they were transported to the site on. The available storage space was in close proximity with the bridge which will make the construction process more efficient.



Figure 29: Google Map Site Design for Storage

The three main areas that will be used for storage are identified in Figure 29 above. Area number one is a vacant lot and is the largest area; it is also the furthest from the construction site. This area will be used to store the heavy machinery that is not used on a daily basis. During demolition, area two will be used to store beams that are being removed from the bridge. Next, these beams will be removed from the site and area two can then be used to store beams awaiting installation. Finally, area three can be used to store any additional materials and also the heavy machinery that is used on a daily basis.

Along with storage, there has to be space for onsite operations. This space is shown below in Figure 30 and Figure 31. Also, when there is construction going on for each side,

there is an off ramp closed from Rt. 20. This closed off ramp can be used to facilitate transport of materials, such as steel beams, to a location close to the bridge.

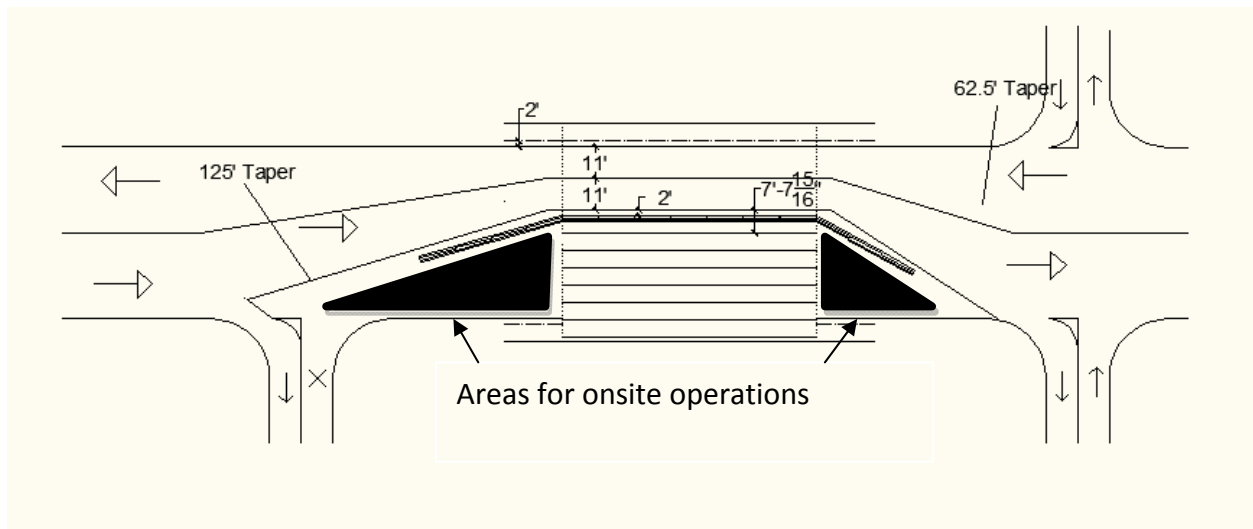


Figure 30: Areas for onsite Operations During Southbound Construction on Rt. 122

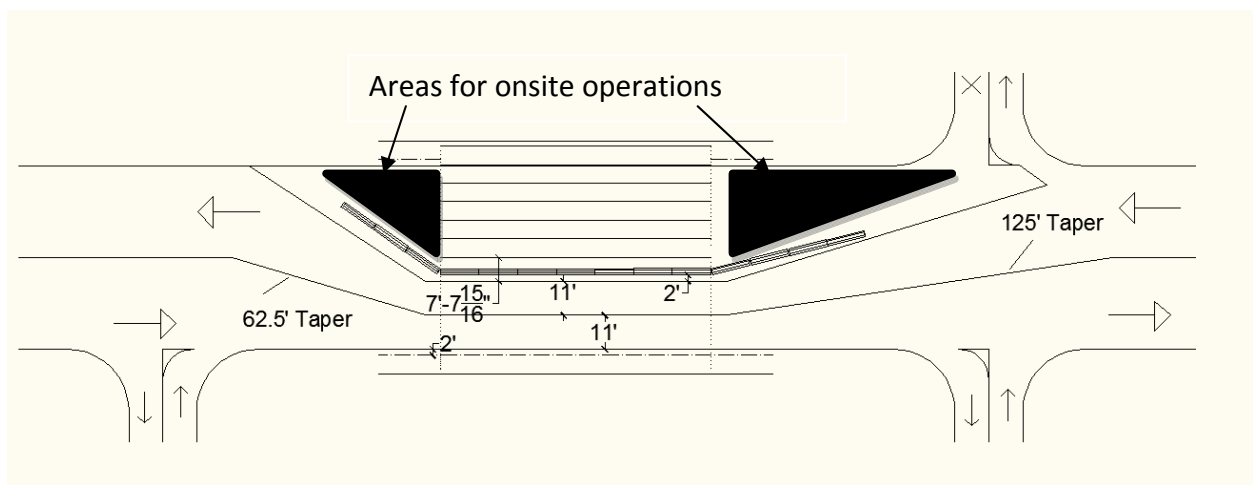


Figure 31: Areas for onsite Operations During Northbound Construction on Rt. 122

5. Conclusions and Recommendations

In this project, we preformed redesign of a typical two lane highway overpass. The basis of the design followed AASHTO specifications and used the LRFD method. To complement our hand calculations, we used Risa-2D to calculate live and dead loads. Along with designing the bridge, we performed an initial and life-cycle cost analysis. In addition to this, we analyzed the construction phase. This included a focus on sustainability, site management, and traffic control.

After completing the full analysis of the Rt. 122 highway overpass, our group has a number of recommendations pertaining to the design and construction of the bridge. These recommendations involve the design of the new bridge, cost analysis, and site management during the construction phase.

5.1 LRFD Design

After following the AASHTO specifications for LRFD bridge design, we decided that the best design for this circumstance would be a composite system. This system would involve 13-W27x102 beams spanning the entire length of the bridge. This system would also include an 8" thick reinforced concrete deck attached to the steel beams by studs.

This system was able to fulfill two of the three goals set by Mass Highway. The first goal set by Mass Highway was to add clearance. Our design is 6" shallower than the original bridge, thus improving the vertical clearance. In addition to this design, we recommend that the existing roadway (Rt. 122) be raised approximately 3". This can be done without negatively impacting the surrounding businesses. These two recommendations combined will increase the clearance by 9", therefore raising the total clearance from 14' to 14'9".

The second goal set by Mass Highway was to increase site distance underneath the overpass. Using a composite system, we were able to span the whole width of Rt. 20 without the need of a center pier. This greatly increased the site distance.

5.2 Cost Analysis

In this project we looked at both the initial and life-cycle cost, and how to minimize these. First, we looked at the initial cost. We calculated that the initial material and construction cost would be about \$670,000. This amount is far less than Mass Highway's project estimation of about \$4.5 million. Although, as stated in the results section, we did not account for every aspect of the construction phase, there still is a large difference between our initial cost and Mass Highway's. This difference shows that the construction and demolition only account for a fraction of the actual initial cost. Also, in an effort to reduce the initial cost of the bridge, we recommend that the contract recycles the steel and concrete in the current bridge.

For life-cycle cost we looked at the cost of a couple different maintenance issues which we knew the bridge would have to go through; repaving, steel painting, concrete repair, asphalt testing, asphalt sealing, and bridge inspection. Over the 75 year life of the bridge, these would cost about \$1.8 million. However, we recommend that Mass Highways puts \$220,000 into an endowment fund with a return of 6%. This initial investment plus its earnings would fund the routine maintenance and periodic painting and repair of the bridge for its 75-year service life.

5.3 Site Management

For site management, there are a number of recommendations we have which will help make the project run smooth and efficient. First, for traffic control we recommend that both

roads stay open. For Rt. 122, we think a lane shift with a few detours for on/off ramps would work the best. In this lane shift, traffic would be condensed onto half the bridge while the other half was rebuilt. In order to protect Rt. 20, we recommend placement of 3X8 boards on the bottom flanges of the bridge with a poly coat on top of them. This will protect cars from falling debris and allow the road to stay completely open during the construction phase.

Along with traffic control, we looked for storage when the construction was taking place. For storing heavy equipment and building materials, there are a number of islands and empty lots that have sufficient space. These are outlined in the results section above.

Sources

American Institute of Steel Construction. (2006). *AISC Steel Construction Manual*. AISC.

Florida DOT. (2009, April 7). *Bridge Costs*. Retrieved from www.dot.state.fl.us/planning/policy/costs/bridges.pdf

Mass Highway. (2009). *Massachusetts Highway Department*. Retrieved September 15th, 2009, from www.mhd.state.ma.us: http://www.mhd.state.ma.us/traffic.asp?f=&C=RTE.%2020

AASHTO LRFD Design Specifications. (1998). Washington D.C.: American Association of State Highway and Transportation Officials.

American Association of State Highway and Transportation Officials. (2004). *A Policy on Geometric Design of Highways and Streets*. Washington, D.C.

Association, F. H. (2006, July 28). *Bridge Technology*. Retrieved from <http://www.fhwa.dot.gov/BRIDGE/lrfd/pscustoc.htm>

Carmichael, A. (2009, November 10). (J. carmichael, Interviewer)

Chen, W.-F. (. (1999). *Bridge Engineering Handbook*. Boca Raton, Florida: CRC Press.

Concrete Network. (2009). *Overview*. Retrieved November 4, 2009, from www.concretenetwork.com: http://www.concretenetwork.com/concrete/demolition/recycling_concrete.htm

Concrete Recycling. (2009). *Economic Feasibility*. Retrieved November 2, 2009, from concreterecycling.org: http://concreterecycling.org/economics.html

Eberle, G. F. (2003, May 1). *Maryland Department of Inspections and Permits*. Retrieved from <http://www.aacounty.org/IP/Resources/PWAUnitPriceList.pdf>

Erico. (2009). *Reinforcing Steel bars*. Retrieved October 12, 2009, from www.erico.com:
www.erico.com/public/library/Concrete/LT0746.pdf
Forecaster, F. T. (2009). *Inflation Data*. Retrieved from
http://inflationdata.com/inflation/inflation_Rate/historicalinflation.aspx

Mass Highway. (2009, August 21). *Design Criteria for MassHighway Projects and*. Retrieved September 28, 2009, from www.mhd.ma.us:
<http://www.mhd.state.ma.us/downloads/engineeringDirectives/2009/E-09-005.pdf>

Mass Highway. (2008, January 30). *Work Zone Safety*. Retrieved September 2009, 2009, from www.mhd.us.ma: <http://www.mhd.state.ma.us/safetytoolbox/downloads/WorkZones.pdf>

Massachusetts Highway Department . (2009). Worcester: Massachusetts Highway Department.

McCormac, J. (2007). *Structural Steel Design, 4th edition*. New York: Prentice Hall.

Meetze, B. (1992, October 8). *South Carolina Department of Highways and Public Transportation*. Retrieved from <http://www.scdot.org/doing/bridge/pdfs/DM199204.PDF>

Nabulsi, M. (2009, September 13). (J. Carmichael, & A. Gager, Interviewers)

Paving, N. (2009, November 18). (A. Gager, Interviewer)

Tonias, D. (1995). *Bridge Engineering*. New York: McGraw-Hill, INC.

U.S. Department of Transportation. (2007, June 14). *mutcd.fhwa.dot.gov*. Retrieved September 27, 2009, from http://mutcd.fhwa.dot.gov/pdfs/2003r1r2/pdf_index.htm

Washington State Department of Transportation. (2006). *Washington State Department of Transportation*. Retrieved September 15, 2009, from
http://www.mhd.state.ma.us/downloads/designGuide/CH_17_a.pdf

Zayed, T. M. (2002, May). *Life-Cycle Cost Based Maintenance Plan for Steel Bridge*. Retrieved November 18, 2009, from <http://pubs.asce.org>:
<http://scitation.aip.org/getpdf/servlet/GetPDFServlet?filetype=pdf&id=JPCFEV000016000002000055000001&idtype=cvips&prog=normal>

Appendix A: Proposal

Redesign and Construction Analysis of Rt. 122 Highway Overpass

Project Number LDA-1003

**Submitted to the Faculty of
Worcester Polytechnic Institute**

By:

Jason Carmichael

Arthur Gager

Date: October 13, 2009

Approved by:

Professor Leonard Albano

Table of Contents

Abstract.....	98
1. Introduction	99
2. Scope and Methodology	100
2.1 Construction Phase	101
2.1.1 On-Site Preparation	103
2.1.2 Traffic Control	104
2.2 Material Selection	105
2.3 LRFD Design	108
2.4 Sustainability	111
2.5 Life-Cycle Cost	113
3. Capstone Design	114
4. Deliverables.....	116
5. Conclusion	118

Table of Figures

Figure 32: Picture of Rt.122 Bridge from Rt.20 (Google Earth).....	99
Figure 33: Arial view of the bridge and surrounding area (Google Earth).....	100
Figure 34: Construction Phase Flow Chart.....	102
Figure 35: Surrounding area showing storage locations (Google Earth).....	104
Figure 36: Construction Phase Methodology	105
Figure 37: Material options.....	106
Figure 38: Framework for material selection.....	106
Figure 39: Material Selection Methodology	108
Figure 40: LRFD Design Flow Chart	109
Figure 41: LRFD Design Methodology.....	110
Figure 42: Sustainability Flow Chart.....	111
Figure 43: Sustainability Methodology	112
Figure 44: Life-Cycle Cost Analysis Flow Chart	113
Figure 45: Life-Cycle Cost Analysis Methodology.....	114
Figure 46: Project Schedule	117

Abstract

This project will study the redesign of a typical two lane highway overpass. The basis of the design will follow AASHTO specifications and use the LRFD method. To complement our hand calculations, we will use Risa-2D to calculate all live and dead loads. Along with designing the bridge, we will perform a life-cycle cost analysis which will include environmental impact, construction practices, and material selections. The final aspect of the project will be to compare our results to the design and cost Mass Highway has determined for this project

1. Introduction

Mass Highway has a number of different bridges and overpasses that are in need of new design and construction in the Worcester area. One of these bridges is a two-span overpass which is part of Grafton Street in Worcester. This overpass is approximately 88 feet from abutment to abutment with steel girders and a reinforced concrete center pier. Two design alternatives will be analyzed based on such factors as initial cost, constructability, life-cycle cost and maintenance. Such design ideas as making it a one-span bridge and what material to use in construction, concrete or steel, will be further investigated. After choosing one of the two alternatives, a final design of the bridge including girders, columns, abutments and the deck will be created. Finally, an estimate for materials will also be made using Materials 3D and unit prices derived from industry standards.



Figure 32: Picture of Rt.122 Bridge from Rt.20 (Google Earth)



Figure 33: Aerial view of the bridge and surrounding area (Google Earth)

2. Scope and Methodology

This project consists of five main components. The first step is to analyze the construction phase, including traffic management, and determine how it can affect the remaining components of our project. The next step is to decide on a material to be used in our primary members. This will be done by analyzing the final three components of our project which are life-cycle cost, sustainability, and bridge design (LRFD format). All of these components will directly affect the material selection phase.

2.1 Construction Phase

The first step is to analyze the construction phase, which according to Mass Highway, is scheduled to begin within the next 3 to 4 years. The construction phase will be particularly complex due to the area. The overpass is on Rt. 122 and extends over Rt. 20 which is a heavily traveled industrial road. There is limited space for material and machinery storage. Also, we will have to look into traffic control and possible detours. Finally, a careful analysis of the construction phase will help us with different components of the design. For example, we will investigate whether it would be possible to keep Rt. 122 open during demolition and reconstruction of the bridge. In order to do this, there would have to be enough space on part of the bridge to support the total traffic flow while the other parts of the bridge were being demolished and rebuilt. Overall, the construction phase will be the most complex portion of the project due to the fact that it directly affects our final design. Figure 3 below outlines the scope of the construction phase.

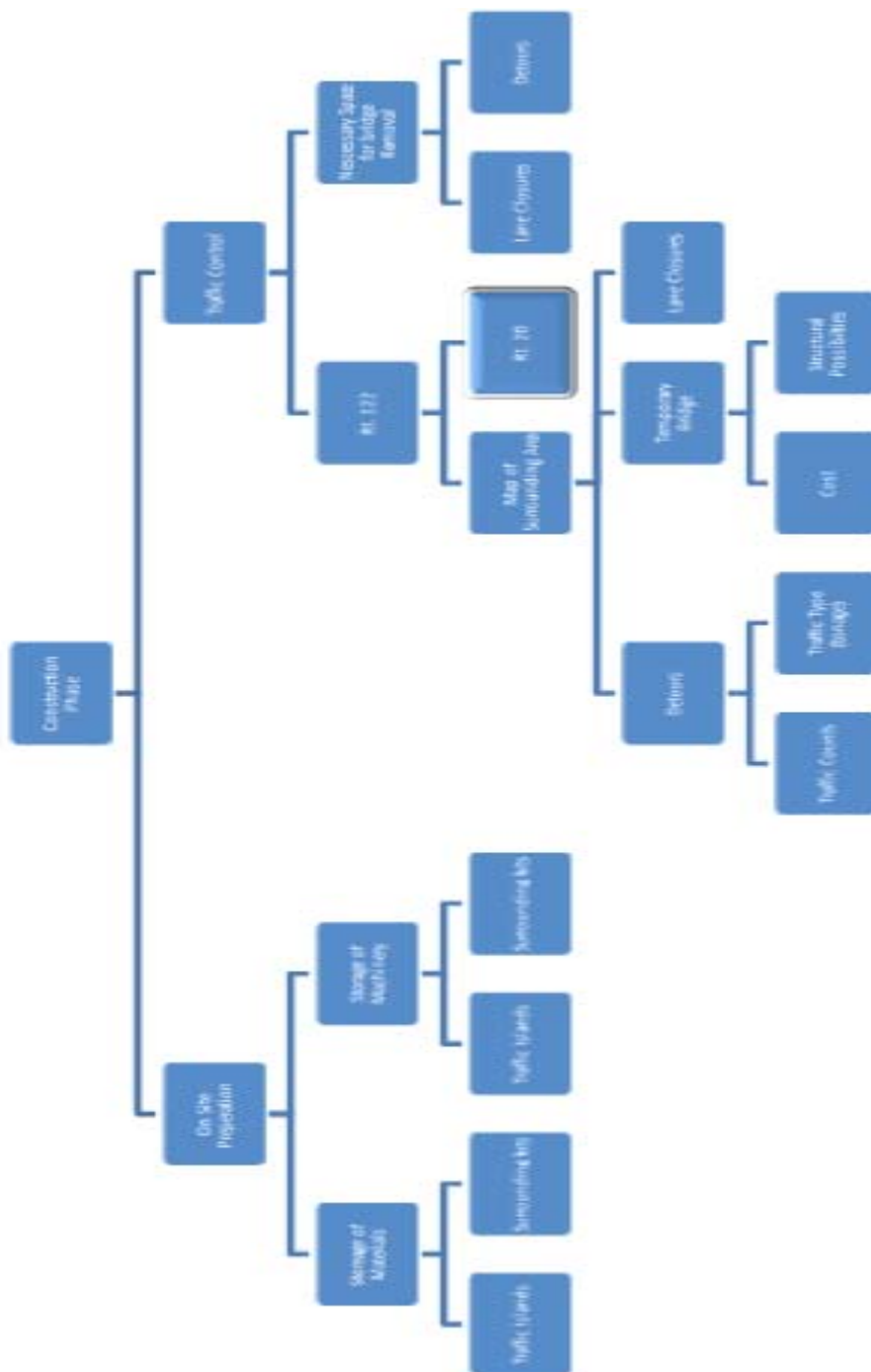


Figure 34: Construction Phase Flow Chart

2.1.1 On-Site Preparation

Having a site that is prepared for the construction phase will both save money and cut down the time needed to complete the job. The site must be suitable to store both materials and machinery being used. For example, if our group chooses to design a one-span steel bridge, the prefabricated bridge structures would have to be shipped to the site. If there was not sufficient storage for these materials then they would have to be shipped in one at a time and placed directly on the supportive abutments. This would slow down the construction phase.

This particular site has limited storage space. The space includes a few traffic islands along with a vacant industrial park. Our group will have to determine how much space will be necessary to store and access the materials and machinery. Before this can be calculated, we will have to choose the materials, steel or concrete, which will be used in the construction of the bridge. Due to the limited space, our group will have to take measurements of all the islands and see if it would be possible to use the industrial park for the storage of the heavy machinery. The location of these areas can be seen below in figure 4.



Figure 35: Surrounding area showing storage locations (Google Earth)

2.1.2 Traffic Control

Another important aspect of the construction phase is traffic control. The area where Route 122 and Route 20 intersect is highly traveled. The plan calls for a complete reconstruction of the bridge. Therefore, our group will have to use Google earth and other maps to determine effective detours for the traffic in this area. When doing so, we will have to take the residential road limitations into account. For example, the tonnage on a busy industrial road such as Route 20 may exceed the tonnage of small residential roads. We will also look at keeping Rt. 122 open during construction. Detours may not be necessary. Our group will also get traffic numbers from Mass Highway. Finally, Route 20 East and Route 20 West are both two-lane roads. Our group will look into possible traffic control options where both lanes remain open or only one lane is closed each direction

during the construction phase. Overall, we will have to map out possible detours taking all restraints into account. Figure 2.4 below outlines the methods that will be used to organize the construction phase of this project.

Construction Phase	
Storage of Materials	Space required to store materials, available space: islands and adjacent lots
Storage of Machinery	Space required to store machinery, type of machinery being used, available space: islands and adjacent lots
Traffic control: Rt. 20	Traffic flow, roadway space available during construction: room required to remove overhead bridge, routes of possible detours: Google earth
Rt. 122	Traffic flow, roadway space available during construction: minimum size of bridge required for traffic flow, routes of possible detours: Google earth, feasibility of temporary bridge

Figure 36: Construction Phase Methodology

2.2 Material Selection

Once we determine a general idea of site layout, we will choose our material. We need to know if there are any factors, such as storage space and traffic flow, which could adversely affect our material selection. This will greatly affect the constructability of each material. For instance, if the site is very tight for space, using large, precast pieces would be difficult. Material selection is a step that has to be done early on because each material has specific characteristics which can affect the whole project. The materials which will be analyzed are outlined in figure 6. In addition to site layout/constructability there are other constraints which will help us in our material selection process. These constraints and how they interact with each other are shown in figure 7.

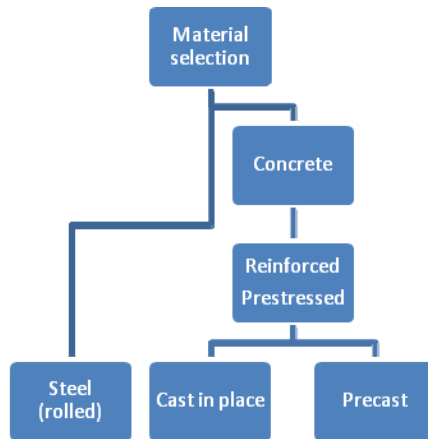


Figure 37: Material options

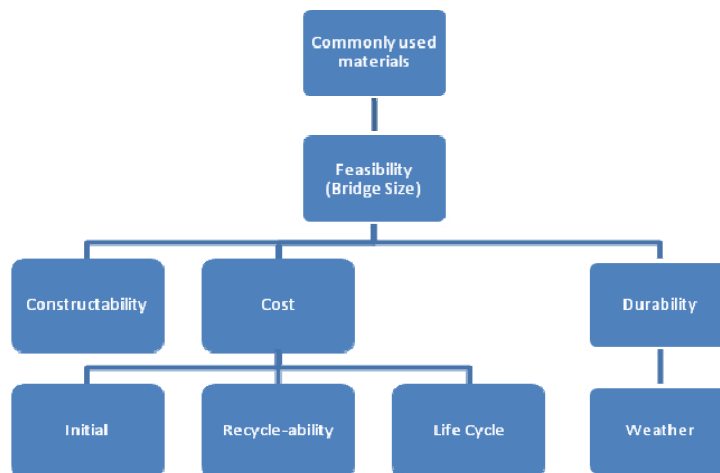


Figure 38: Framework for material selection

We are first going to look at the commonly used building materials for bridges in the North East. Determining common materials is important because every region has its own specific constraints. Along with looking at the commonly used materials, we will also have to look at what types of bridges these materials support and compare this to the bridge we are trying to build. Small bridges that support lesser loads can use a far wider range of materials. From these materials, we are then going to look at specific characteristics of each material.

There are three basic qualities which we think are necessary to consider when choosing a material. These are constructability, durability, and cost (initial and life-cycle). When looking at

constructability we are going to see how easily it is to assemble a bridge made out of this material and if it's desirable with our site layout. For example, this bridge is in an urban area, and we need to choose a material that can be installed with minimal affects on traffic flow. Next we are going to look at the material's durability. This is affected by the road usage and the weather conditions. For instance, salts contribute to corrosion of building materials which makes them extremely weak, so we must look at this. Also, heavy traffic can wear down some materials much faster, specifically the wearing surface.

The last aspect we are going to look at is cost. This will be done through different methods. We are going to look at how much the material initially costs, and analyze life-cycle costs. We need to make sure that even though a material costs less to install, it doesn't end up costing more over the bridge's lifetime through maintenance.

Figure 8, below outlines the steps that we will take in order to select the material for each component of the overpass. Each one of these steps outlines a different characteristic for the building materials. When deciding which material to use, emphasis will be put on feasibility which includes the material's ability to support necessary loads while eliminating the issues of the current bridge, such as low clearance. It is possible that more than one material will be able to meet these limitations. Therefore, we will also look at the constructability, durability, and cost. Overall, the quantitative results of feasibility and cost will be matched up with the qualitative results of constructability and durability in order to make a final decision.

Once we have determined our material, we are going to look to see if there is a way we can get this material through a recycling process. This comes second hand to cost due to the fact that cost is the main issue when it comes to state projects. We are going to see the availability and cost of using recycled materials and weigh that against using non-recycled materials. Then,

if it doesn't cost more, we will try and use it for our project. Figure 8 outlines the methodology we will use for our material selection.

Material Selection	
Material List	Which materials can hold up to weather and typical size limitations for a bridge-reinforced concrete, pre-stressed concrete, steel
Feasibility	Each material's strength, Bridge loads, Span Size
Constructability	Space/ prep area needed to install material, set up limitations
Durability	Ability to hold up in bridge's conditions, weather, corrosion, usage
Cost	Estimate how much it to build out of the material, maintenance of the material
Recyclability	Availability of recycled material, cost of recycled material

Figure 39: Material Selection Methodology

2.3 LRFD Design

Our group plans on using the LRFD design method, which is within the AASHTO specifications, in creating the new bridge. We are going to use this method to create two different designs, a single and a two-span. Then we are going to look at the ability of these to fix issues with the current bridge and use this determine a final design. Figure 9 below outlines the scope of design.

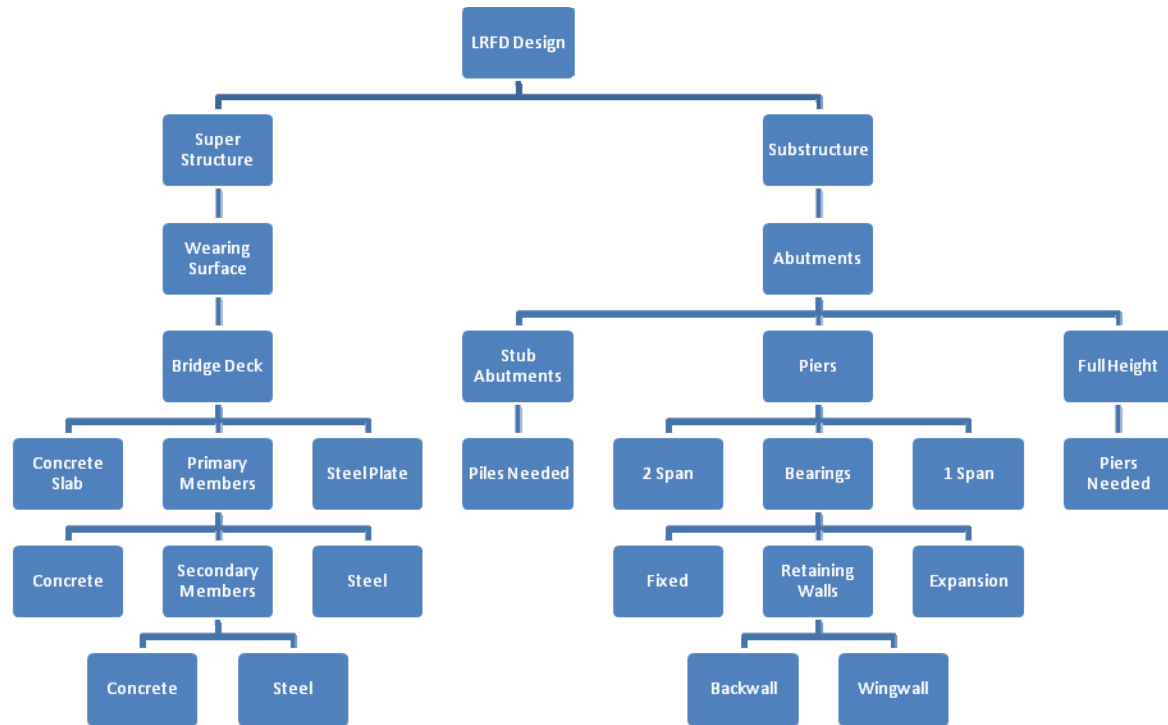


Figure 40: LRFD Design Flow Chart

There are two main sections to every type of bridge and overpass, which are the superstructure and substructure. The superstructure contains all of the most visible portions of the bridge. These include the wearing surface, deck, primary structural members and secondary structural members. There are decisions that have to be made for each of these components. Our group will have to decide between having concrete slabs or steel plates for our bridge deck. Also, we will have to choose to use either concrete or steel in the design of our primary and secondary members. The main factor in choosing a material will be its ability to meet the goals of the new bridge, such as clearance. In addition to this, we will consider cost, durability and constructability.

The substructure acts as the foundation of the bridge. The substructure is composed of abutments, piers, bearings and retaining structures. Decisions will have to be made regarding each component of the substructure as well. Our group will look at the differences between a single and two-span bridge. We will also have to choose between the types of abutments to use. There are two main types of abutments. These are full-height and stub abutments. From there, we will have to decide, through research, whether piles will be necessary to support these abutments. This research will include looking at soil profiles and the current bridge abutments. We will also have to choose the type of bearings to use to distribute the vertical load. The two main types of bearings are expansion and fixed bearings. Finally, we will have to analyze the site to make a decision on what type of retaining structures to use.

Overall, by calculating all live and dead loads and using LRFD design, we will design each component of the superstructure and substructure separately. Once this is done, we will compare the differences between having a one-span and two-span overpass. Comparisons will include cost, time and maintenance. Then we will use RISA to help check our calculations.

Figure 10 below outlines the methods that will be used.

LRFD Design	reasons for redesigning, center pier problems, bridge profile, surrounding land use, dead and live loads(Risa-2D)
Superstructure	Bridge width, number of lanes, shoulders, side walks
Wearing Surface	Thickness, road usage, traffic counts, weathering conditions-freeze thaw
Bridge Deck	Thickness, overhang, design criteria (LRFD format), material strength: concrete and reinforcing steel
Primary Members	Material, number of girders, spacing of girders, type of support; composite beam, connection type, on site fabrication, LRFD Design steps
Substructure	Type of abutments, necessity of earth support (Retaining walls), Bearing type, LRFD design steps

Figure 41: LRFD Design Methodology

2.4 Sustainability

Along with just looking at the basic design of this bridge, we will try to consider sustainability. However, we know that a large part of projects of this nature is the cost. So, we will try to focus on this. There are many different ways which money can be saved while being ecologically friendly. The first part will be determining what current practices are. We will need to see what is already being done for similar jobs to see what the ability is to improve on this.

Figure 11 below illustrates the different paths we will be considering for sustainability.

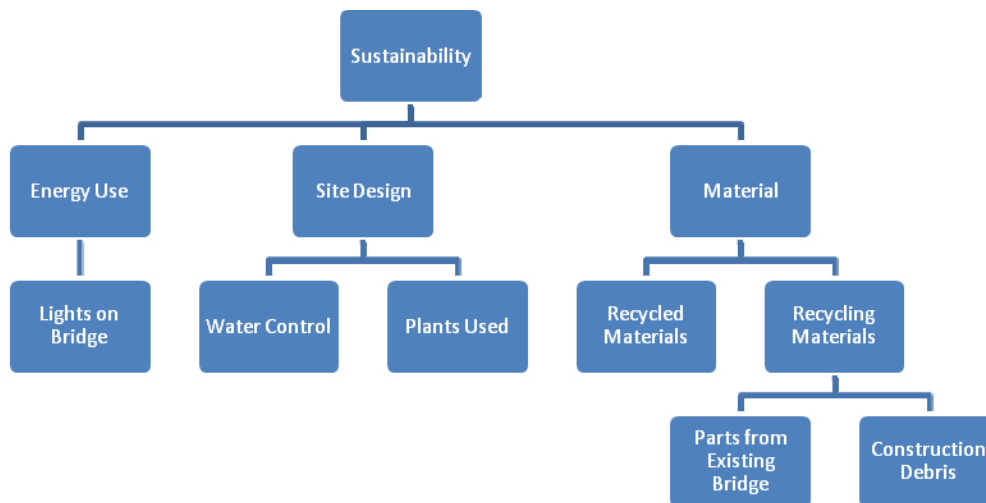


Figure 42: Sustainability Flow Chart

We will first focus on the initial environmental impact of this project. This is affected by the construction materials and debris. We are going to look into the possibility of using recycled materials for the members of the bridge. This could be anything from using steel beams with a higher percentage of recycled steel, or using a different type of road material. Also, we are going to look into recycling material. We will see what the benefits are of recycling construction debris and parts of the demolished existing bridge.

Another aspect we are going to look at is long term sustainable practices which can be employed on this bridge. One of the most obvious ways to save money is through electrical use. We are going to determine if lights are required for this bridge, and if they are, we will see if there are ways to save money powering these. We will also look at site design. We are going to see if there are ways to save money in maintenance through water control. Along with water control for the site, if local plants are used once the project is finished, there could be many financial benefits. First, being local, they are more common and cost less, and they will have a better chance to live and prevent erosion.

Once have determined these methods, we are going to compare them to the LEED certification standards. LEED certification is a program created by the USGBC to rate building projects on their environmental impact. It is mostly used for new and remodeled office and residential buildings. However, there are some areas which our project could overlap with these. These areas include electrical use, site design, water runoff, and material use. Figure 12 illustrates the methods we will be using to determine the best approach for design when considering sustainability.

Sustainability	focus on cost, initial and long term
Limit Energy Use	Common practice, possible improvements, lights: LED, solar
Water Control	Rainfall amounts, common practices, chance for improvement, storm water run off, erosion
Material Used	Availability of recycled material, cost of recycled material
Limit Wasted Material	Common practice, availability of recycling material, cost of recycling material

Figure 43: Sustainability Methodology

2.5 Life-Cycle Cost

In any public works project, the cost is a very important factor. Along with initial cost, it is important to look at how much it will cost to keep whatever is built in working order throughout its lifespan. This is why we are going to do a life-cycle cost analysis once the whole bridge is built. Figure 2.9 below illustrates the scope of our life-cycle cost analysis.

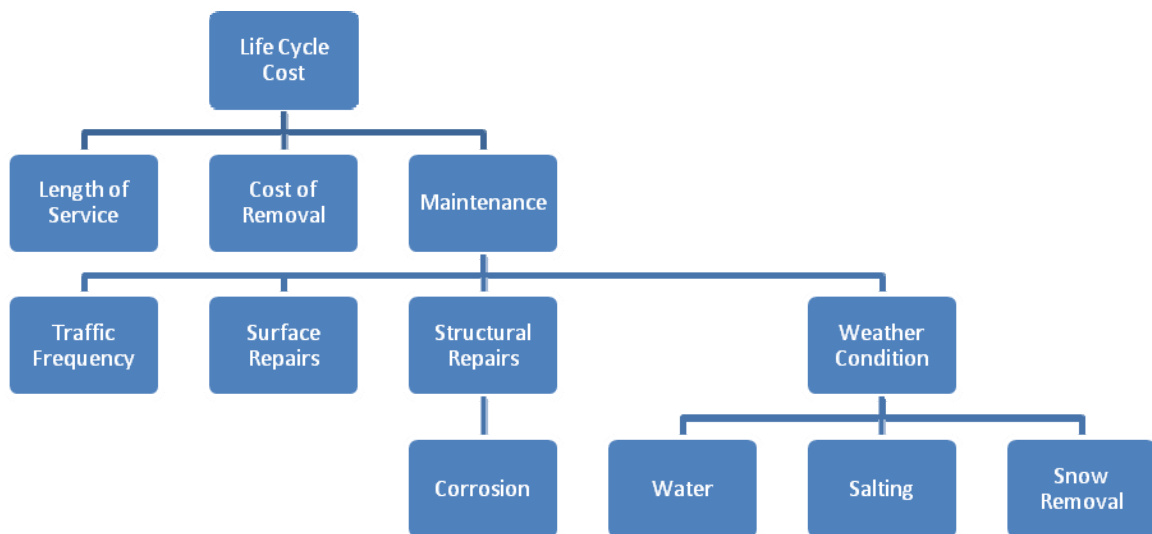


Figure 44: Life-Cycle Cost Analysis Flow Chart

We are going to look at three main parts. These are the length of service of the bridge, the cost for removal of the material, and the cost to maintain the bridge. In this life-cycle analysis, we are going to figure out how long the bridge will be in use and when it will start to cost more to maintain the bridge than it is actually worth. This will be done by looking at other bridges made out of the same materials with the same types of maintenance issues. Finding this out will help us determine how long the life-cycle actually is. Along with the time of use, we need to figure out what will be done with the bridge once its use is up.

The most important part of the bridge's life-cycle cost is maintenance. This is because it can cost more to maintain a bridge than the bridge actually cost to build. Also, this bridge is a very important part of the region's infrastructure so it must be kept safe. Bridge maintenance is a broad title which has many different parts. We plan on looking at the maintenance of other bridges with similar traffic patterns to help us calculate the maintenance on this bridge. Next we will look at the surface repairs. We need to see how often it will have to be resurfaced. Then we will look at the structural repairs. A large factor we have to look at is corrosion prevention and repairs. No matter if we use rolled steel or reinforced concrete, steel will be used, which can corrode.

Both the structural and surface repairs are greatly influenced by weather conditions. The salts from winter will corrode steel being used and the water in spring will cause erosion around the footings. We will have to see to what extent these things affect the bridge and the actual cost they add to the lifecycle of the bridge. Figure 14 below illustrates the methods we will use when perform our life-cycle cost analysis.

Life cycle Cost	
Service Life	Life Expectancy of the bridge
Cost of removal	money to remove bridge, any hazardous materials, any recyclable materials
Maintenance	Types of repairs: painting, waterproofing, join repairs, resurfacing, etc.
	Frequency of repairs on similar bridges
	Cost of repairs on similar bridges

Figure 45: Life-Cycle Cost Analysis Methodology

3. Capstone Design

Along with completing our Major Qualifying Project, we must complete our Capstone Design Requirement. This capstone design requirement will follow the ABET guidelines and

show our ability to design a system which will include economic, environmental, social, health and safety, and sustainability aspects (Commission, 2007).

One goal of our project is to perform a cost analysis for both the initial construction and the life-cycle. This cost analysis will show the economical impact of this bridge. It will also give us an understanding on how different aspects of a construction project can affect the cost of a large scale project.

While we are designing this bridge, we are also going to keep in mind the environmental impact of this project. This will be done by lessening the initial impact the project has on resources. We are going to look into using recycled building materials and recycling construction debris.

This bridge will have a large social aspect. Along with hopefully having less of an impact on the environment, this will affect the common people because it is necessary for traffic flow. Once this bridge is completed, it will improve the flow of traffic. Also, during construction, we are going to have to constantly keep in mind the affect the work has on passing motorist, bicyclists, and pedestrians (Highway, 2009).

In the design of the Rt. 122 bridge, we will also consider public health and safety. This bridge will also follow AASHTO LRFD design, which is commonly used to design bridges. This system uses load and resistance factors to ensure a stable bridge with sufficient strength. In addition to this, during construction of the bridge, we will establish practices for traffic control in order to protect motorists, bicyclists, pedestrians, and construction workers on the site. Also, the current bridge has issues which make it unsafe to people who use it, such as low visibility and low clearance. We are going to look into ways at making these issues less prevalent.

Lastly, we are going to focus on sustainability. Sustainability is the ability to meet the needs of the present without compromising the ability of future generations to meet their own needs (Commission, 2007). We will look at site design. One way to help with sustainability is through storm water management. We are also going to look at using local plants in the landscaping of the area once the bridge is complete. We will also consider different ways to manage storm water runoff.

4. Deliverables

Overall, our group will be doing a full analysis of the design of the Rt. 122 overpass in Worcester. Our project will include designs of each component of the overpass which will be hand calculated using the LRFD method, then checked using RISA. Also, we will do a life-cycle cost analysis. Finally, the construction phase will be the most complex portion of our project. The main issue to be resolved will be traffic control issues. Our group will propose a traffic control plan while keeping in mind that both Rt. 122 and Rt. 20 are highly traveled roadways. This traffic control plan will be discussed in depth and depicted through the appropriate tables and figures. Finally, we will draw up a schematic for both our traffic plan and the design of our bridge using AutoCad. Below is the schedule which shows the time period that we will be completing each of our project objectives.

	Week															
Goals	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Site examination																
Proposal																
Abstract																
Introduction																
Scope																
Capstone Design																
Methodology																
Conclusion																
Design																
Material Research																
Super Structure																
Substructure																
Cost Estimator																
Construction Phase																
Traffic Control																
On site Prep																
Sustainability																
LEED standard Research																
Material Recycling																
Life Cycle Cost analysis																
Writing MQP																

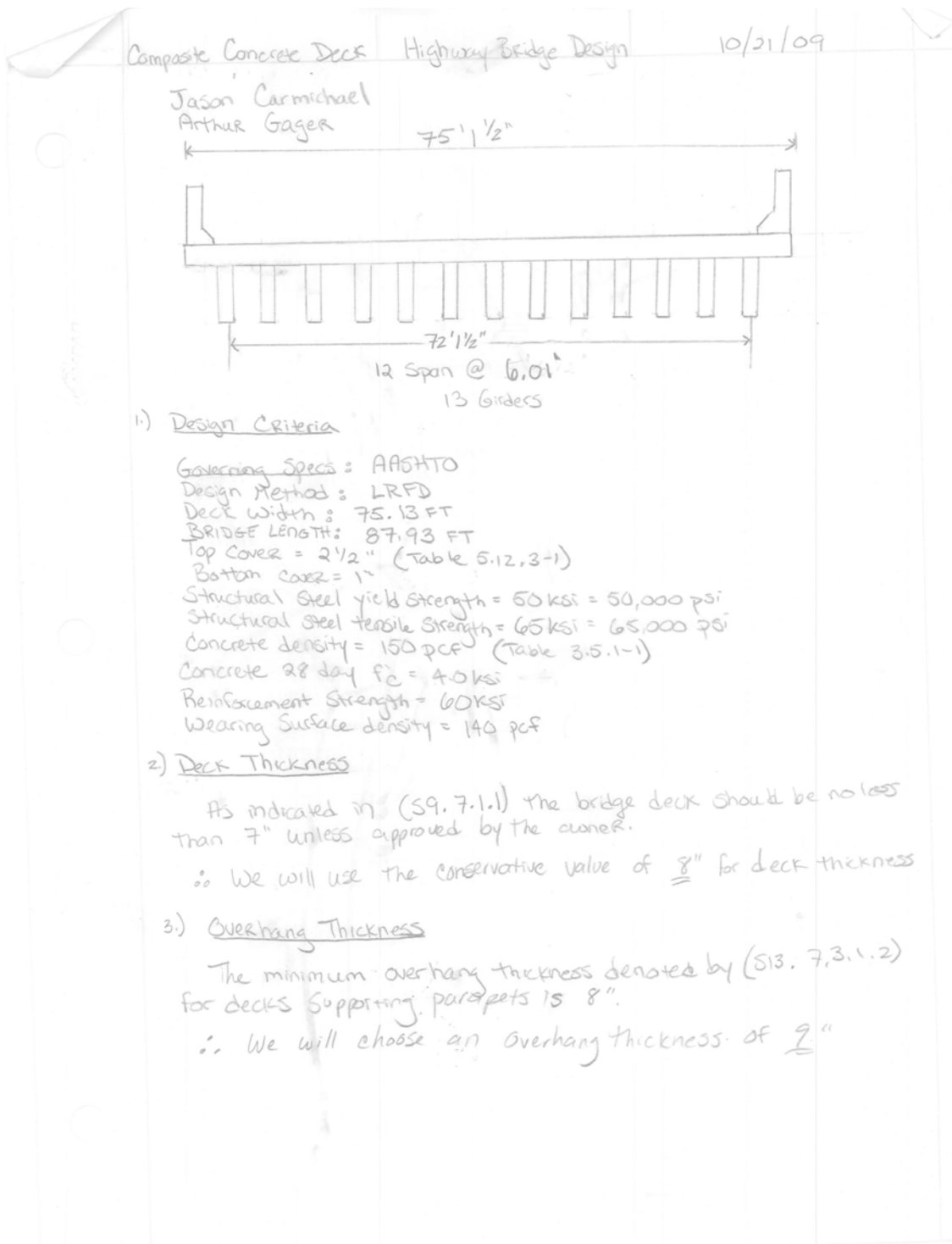
Figure 46: Project Schedule

5. Conclusion

In our project, we are going to redesign the Rt. 122 Bridge which extends over Rt.20. We plan on covering every aspect of the redesign. This is going to include the design of the actual replacement bridge, the affect this bridge will have on the surrounding area through an environmental impact, and the logistics associated with the construction phase.

In completing this project, we are going to have to use a number of tools. We will have to get bridge history reports in order to see the deficiencies of the current bridge, including height issues and pier quality. We are also going to have to determine what the ASHTO design standards are and apply them to this bridge. Through these events, along with others, we expect to get a good understanding of the construction phase and end up with a product similar to what was designed and approved by Mass Highway for this bridge.

Appendix B: Composite Deck Design



Composite Concrete Deck

2.

4) Parapet Selection / Design

- Our group chose to use a type F parapet.

Values / Dimensions (LRFD Design Example, FHWA)

Mass per unit length = 650 lb/ft.

Width at base = 1' 8 1/4"

Moment capacity at base (M_c/length) = 17.83 k-ft/ft.

Height = 42"

Length of parapet failure mechanism $L_c = 235.2"$

Collision load capacity = 137.22 k

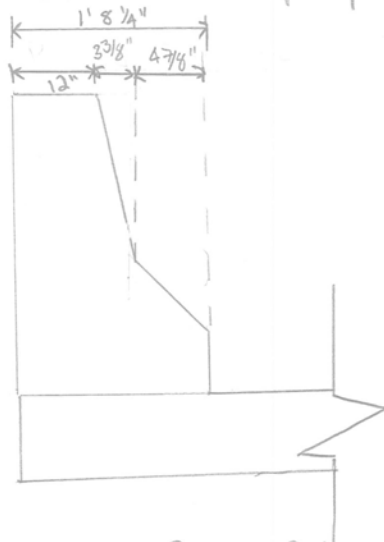


Figure : Parapet Schematic

5) Design dead load moments

All load factors below are from; 53.4.1

Slab and Parapet

min = 0.9

max = 1.25

Future Wearing Surface (FWS)

min = 0.65

max = 1.5

$$\text{Self weight of deck} = t(d_c)/12 = 8(150)/12 = \underline{\underline{100 \text{ psf}}}$$

Unfactored Dead Load Moments

$$M = wL^2/c$$

- Unfactored self weight positive/negative moment = $(100/1000)(5.55)^2/10$
 $= 0.31 \text{ k-ft/ft}$
- Unfactored FWS +/- moment = $(30/1000)(5.55)^2/10$
 $= \underline{0.092 \text{ k-ft/ft}}$

6.) Determine live load Effects

Method of deck analysis denoted by, S4.6.2

Given:

- minimum distance from center of wheel to inside face of parapet = 1 ft (S3.6.1.3)
- minimum distance between wheels of 2-adjacent trucks = 4 ft.
- Dynamic load allowance = 33% (S3.6.2.1)
- Load Factor = 1.75 (S3.4.1)
- Multiple presence factor (S3.6.1.1.2)
 - single lane = 1.20
 - 2-lanes = 1.00
 - 3-lanes = 0.85
- Resistance factors, ϕ for moments:
 - $\phi_{\text{strength}} = 0.90$ (S5.5.4.2)
 - $\phi_{\text{ext}} = 1.00$ (S1.3.2.1)

7.) Positive Moment Design in Deck

- From Table SA4.1-1, for girder spacing of 6.01'

→ This value will be obtained assuming a girder spacing = 6.0' due to the fact that using interpolation will be insignificant

∴ For girder spacing of 6', unfactored live load "+" moment per unit width =

$$\boxed{4.83 \text{ k-ft/ft}}$$

- Maximum factored "+" moment per unit width = 1.75 (4.83)

$$M_{u+LL} = \boxed{8.45 \text{ k-ft/ft}}$$

- This moment is applicable to all positive moment regions in all bays of the deck. (S4.6.2.1.1)

8) Unfactored dead load Moments

* Our group utilized RISA 2D to calculate the Maximum (M_u) positive and negative moments for the deck slab, parapet, and future wearing surface.

Deck Slab

$$M_{u+} = 1.558 \text{ K/FT}$$

$$M_{u-} = -3.116 \text{ K/FT}$$

Parapet

$$M_{u+} = 1.073 \text{ K/FT}$$

$$M_{u-} = -2.146 \text{ K/FT}$$

Future Wearing Surface

$$M_{u+} = 0.466 \text{ K/FT}$$

$$M_{u-} = -0.932 \text{ K/FT}$$

9) Factored Maximum Dead Load Moment (+)

Given: Load factors from (S3.4.1)

<u>Slab</u>	<u>Parapet</u>	<u>FWS</u>
1.25	1.25	1.50

$$M_{u_{DL}} = 1.25(M_{u_{deck}}) + 1.25(M_{u_{parapet}}) + 1.50(M_{u_{FWS}})$$

$$M_{u_{DL}} = 1.25(1.558) + 1.25(1.073) + 1.50(0.466)$$

$$M_{u_{DL}} = 1.948 + 1.341 + 0.699$$

$$M_{u_{DL}} = 3.988 \text{ K-FT/FT}$$

10) Total Factored Design Moment (+)

$$M_{u_{TOTAL}} = M_{u_{DL}} + M_{u_{LL}}$$

$$M_{u_{TOTAL}} = 3.988 + 8.45 = 12.44 \text{ K-FT/FT}$$

Composite Concrete Deck

5

11.) Utilize TABLE A4.1-1 For UNfactored Live Load (-)

- According to table A4.1-1, for a girder spacing of 6' and a distance of 6" from center line of girder to design section the maximum live load moment per unit width is 3.50

→ Once again, we will assume that 6.01' is equal to 6' for girder spacing to make the problem simpler.

∴ For a girder spacing of 6', the maximum negative live load moment (M_{uLL}) = 3.50

12.) Maximum Factored Live Load Moment (-)

Given: Live Load Factor = 1.75 (S3.4.1)

$$\therefore M_{uLL} = 1.75(3.50) = \boxed{-6.125 \text{ K}\cdot\text{FT}/\text{FT}}$$

13.) Factored Maximum Dead Load Moment (-)

Given: Load Factors from (S3.4.1)

Slab	Parapet	FWS
0.9	0.9	0.65

$$M_{uDL} = 0.9(-M_{u\text{deck}}) + 0.9(-M_{u\text{parapet}}) - 0.65(M_{u\text{FWS}})$$

$$M_{uDL} = 0.9(-3.116) + 0.9(-2.146) - 0.65(-0.932)$$

$$M_{uDL} = \boxed{-5.34 \text{ K}\cdot\text{FT}/\text{FT}}$$

14.) Total Factored Design Moment (-)

$$M_{u\text{TOTAL}} = M_{uLL} + M_{uDL}$$

$$M_{u\text{TOTAL}} = -6.125 - 5.34$$

$$\boxed{M_{u\text{TOTAL}} = -11.47 \text{ K}\cdot\text{FT}/\text{FT}}$$

Composite Concrete Deck

6

15.) Design For Positive Moment in Deck CONT'D

Assume #5 bars w/ ϕ (diameter) = 0.625 in
 a (area) = 0.31 in²

$$R_n = \frac{M_{u(TOTAL)} (b)}{(\phi \cdot b \cdot d_e^2)}$$

given: $b = 12$ in.
 $\phi = 0.9$

• Must solve for d_e

d_e = Effective depth

d_e = Total thickness - bottom cover - $\frac{1}{2}$ bar ϕ - integral wearing surface

$$d_e = 8 - 1 - \frac{1}{2}(0.625) - 0.5$$

$$d_e = \underline{6.19 \text{ in}}$$

$$R_n = \frac{(12.44)12}{(0.9)(12)(6.19)^2}$$

$$R_n = \frac{147}{413.81} = \underline{0.361 \text{ K/in}^2}$$

$$P = 0.85 \left(\frac{f'_c}{f_y} \right) \left[1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right]$$

$$P = 0.85 \left(\frac{4}{60} \right) \left[1 - \sqrt{1 - \frac{2(0.361)}{0.85(4)}} \right]$$

$$P = \underline{0.0063}$$

$$A_s = P d_e$$

$$A_s = (0.0063)(6.19) = 0.039$$

16.) Determine Bar Spacing

$$\text{Spacing} = \frac{A_{\text{bar}}}{A_{\text{steel}}} = \frac{0.31}{0.039} = 7.95 \text{ in.}$$

\therefore We will Try #5 bars Spaced 7 in

17.) Check Maximum and Minimum Reinforcement

- Note \Rightarrow Reinforced concrete sections are considered under-reinforced when $c/d_e \leq 0.42$. (Don't want design to be over-reinforced)

Calculated Values:

T = Tensile Force in the tensile reinforcement (k)

$$T = 0.31 (60)$$

$$T = 18.6 \text{ k}$$

Given: $f'_c = 4.0 \text{ ksi}$
 $\beta_1 = 0.85$

$$a = \frac{T}{0.85(f'_c)(\text{spacing})}$$

$$a = \frac{18.6}{0.85(4)(7)}$$

$$a = 0.78 \text{ m}$$

$$c = \frac{a}{\beta_1}$$

$$c = \frac{0.78}{0.85} = 0.918 \text{ m}$$

• Check Section For Over-Reinforcement

$$c/d_e = 0.918/6.19$$

$$c/d_e = 0.15 < 0.42 \quad \text{OK} \quad \checkmark$$

18.) Check For Cracking Under Service Limit / State (5.7.3.4)

$$f_{sa} = Z / (d_e A)^{1/3} \leq 0.6 f_y$$

$$0.6 f_y = 0.6 (60 \text{ ksi}) = 36 \text{ ksi}$$

• Calculated Values

d_c = Thickness of concrete cover measured from extreme tension fiber to center of bar located closest thereto. (in)

$$d_c = 1.3125 \text{ OR } 1" + \frac{\text{bar } \phi}{2}$$

$$d_c = \frac{0.625}{2} + 1$$

$$\underline{\underline{d_c = 1.31"}}$$

A = Area of concrete having the same centroid as the principal tensile reinforcement and bounded by the surfaces of the cross-section and a straight line parallel to the neutral axis, divided by the number of bars (in^2)

$$A = 2 d_c (\text{spacing})$$

$$A = 2(1.31)(7")$$

$$\underline{\underline{A = 18.34 \text{ in}^2}}$$

Z = Crack control parameter (in/m)

$Z = 130 \text{ in}/\text{m}$ for severe exposure conditions (de-icing salts/weathering)

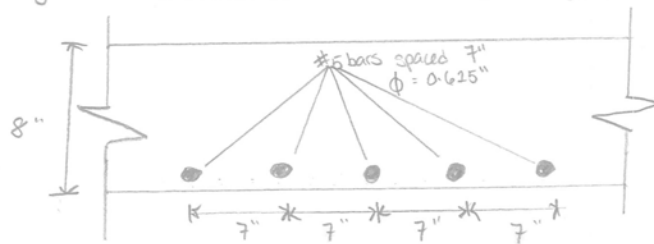
$$\therefore f_{sA} = Z / (d_c A)^{1/3}$$

$$f_{sA} = 130 / [(1.31)(18.34)]^{1/3}$$

$$f_{sA} = 45.1 \text{ ksi which is } \geq 36 \text{ ksi}$$

\therefore Use maximum allowable $\underline{\underline{f_{sA} = 36 \text{ ksi}}}$

Figure: Schematic for Bottom Transverse Reinforcement



Composit Concrete Deck

9

19.) Stresses under Service Loads (+)

Given: Modular ratio for 4ksi concrete $\Rightarrow n = 8$ (56.10.3.1)

Service Live Load Moment (+)

- Once again, referring to Table SA4.1-1 the maximum unfactored positive live load moment for 6' Spacing is 4.83 K-FT/FT , Factored Moment = $1.0(4.83) = \underline{\underline{4.83 \text{ K-FT/FT}}}$

Service dead load Moment (+)

Given: Load Factors for slab, parapet and FWS = 1.0 (3.4.1-1)

- As calculated in Step #8, Max positive moments are:

<u>Slab</u>	<u>Parapet</u>	<u>FWS</u>
1.558 K/FT	1.073 K/FT	0.466 K/FT

- Factored Total Max dead load Moment (+)

$$M_{u_{DL}} = 1.0(1.558) + 1.0(1.073) + 1.0(0.466)$$

$$M_{u_{DL}} = \underline{\underline{3.097 \text{ K-FT/FT}}}$$

- Total Factored Design Moment (+)

$$M_{u_{TOTAL}} = M_{u_{DL}} + M_{u_{LL}}$$

$$M_{u_{TOTAL}} = 3.097 + 4.83$$

$$M_{u_{TOTAL}} = \underline{\underline{7.93 \text{ K-FT/FT}}}$$

- Find TRANSFORMED moment of Inertia/ d_e

- Calculated Values:

d_e = effective depth from compression fiber to centroid of tensile force in the tensile reinforcement (in.)

$$d_e = 8 - 1 - \frac{1}{2}(0.625) - \frac{1}{2} = \underline{\underline{6.19''}}$$

$$n = 8$$

$$A_s = \text{Assumed to be } 0.44 \text{ For This Step. } \left\{ \text{LRFD Design Example} \right\}$$

Composite Concrete Deck

10

$$I_{\text{TRANSFORMED}} = \left(\frac{1}{3}\right)b(k)(d_c)^3 + nA_s(d_c - kd_c)^2$$

∴ MUST Solve For K.

$$k = \frac{\sqrt{(pn)^2 + 2pn} - pn}{n}$$

$$A_s = \rho b d_c$$

$$\rho = \frac{A_s}{b d_c} = \frac{0.74}{12(6.19)}$$

$$\rho = 0.01$$

$$k = \frac{\sqrt{0.01(8)^2 + 2(0.01)(8)} - 0.01(8)}{8}$$

$$k = 0.327 \approx 0.33$$

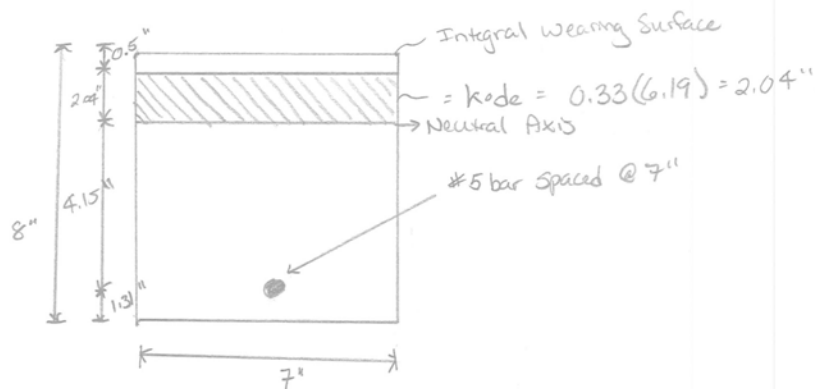
• Now that we have all variables known, we must solve for $I_{\text{TRANSFORMED}}$

$$I_{\text{TRANSFORMED}} = \left(\frac{1}{3}\right)(12)(0.33)(6.19)^3 + 8(0.74)(6.19 - 0.33(6.19))^2$$

$$I_{\text{TRANSFORMED}} = 134.094 + 101.823$$

$$I_{\text{TRANSFORMED}} = 135.92 \frac{\text{in}^4}{\text{ft}}$$

FIGURE : Schematic For Crack Control of Positive Moment Reinforcement Under Line Loads:



Composite Concrete Deck

11

20) Stress in Reinforcement

$$f_s = \frac{n (M_{u_TOTAL} \cdot b \cdot Y)}{I_{TRANSFORMED}}$$

$$M_{u_TOTAL} = 7.93 \text{ K-FT/FT}$$

Y = distance from neutral axis to compression face of section.

$$Y = d_e - k(d_e)$$

$$Y = 6.19 - 0.33(6.19)$$

$$Y = 4.15 \text{ in}$$

$$f_s = \frac{8 (7.93 \cdot 12 \cdot 4.15)}{135.92}$$

$$f_s = 23.24 \text{ ksi} < 36 \text{ ksi (Allowable service load stress)} \checkmark \underline{\text{OK}}$$

21.) Design For Negative Moment in Deck (similar to Step #15)

• Once again, we will assume #5 bars

$$\phi \#5 \text{ bars} = 0.625 \text{ in}$$

$$A \#5 \text{ bars} = 0.31 \text{ in}^2$$

$$R_n = \frac{M_{u(-)TOTAL} (b)}{\phi \cdot b \cdot d_e^2}$$

$$\text{Given: } b = 12 \text{ in} \\ \phi = 0.9$$

• Must solve for d_e

$$M_{u_TOTAL(-)} = -11.26 \text{ K-FT/FT}$$

d_e = Effective depth

d_e = Total Thickness - TOP COVER - $\frac{1}{2}$ bar diameter

$$d_e = 8 - 2.5 - \frac{0.625}{2}$$

$$d_e = 5.19 \text{ in}$$

$$R_n = \frac{11.47(12)}{0.9(12)(5.19)^2}$$

$$R_n = 0.47 \text{ K/in}^2$$

Composite Concrete Deck

12

$$\rho = 0.85 \left(\frac{f'_c}{f_y} \right) \left[1.0 - \sqrt{1.0 - \left(\frac{2RN}{0.85f'_c} \right)} \right]$$

$$f'_c = 4.0 \text{ ksi}$$

$$f_y = 60 \text{ ksi}$$

$$\rho = 0.85 \left(\frac{4}{60} \right) \left[1.0 - \sqrt{1.0 - \left(\frac{2(0.47)}{0.85(4)} \right)} \right]$$

$$\rho = .0083$$

$$A_s = \rho d_e$$

$$A_s = .0083 (5.19)$$

$$A_s = .043$$

22.) Determine Bar Spacing

$$\text{Spacing} = \frac{A_{\text{req}}}{A_{\text{steel}}} = \frac{0.31 \text{ in}^2}{.043 \text{ in}^2} = 7.21 \text{ in}$$

∴ We will Try #5 bars spaced @ 7 in

23.) Check Maximum and Minimum Reinforcement

*Note ⇒ Reinforced concrete sections are considered under-reinforced when $c/d_e \leq 0.42$ (Don't want design to be over-reinforced)

• Calculated Values

$$T = \text{Tensile Force in tensile Reinforcement (k)} \quad \text{Given: } f'_c = 4.0 \text{ ksi} \quad \beta_1 = 0.85$$

$$T = 0.31 (60)$$

$$T = 18.6 \text{ k}$$

$$a = \frac{T}{0.85(f'_c)(\text{Spacing})}$$

$$a = \frac{18.6}{0.85(4)(7)}$$

$$a = 0.78 \text{ in}$$

$$c = \frac{a}{\beta_1}$$

$$c = \frac{0.78}{0.85}$$

$$c = 0.92$$

Composite Concrete Deck

13

- Check Section For over-Reinforcement

$$c/d_e = 0.92/5.19$$

$$c/d_e = 0.18 \leq .42 \quad \checkmark \underline{\underline{OK}}$$

- 23.) Check For Cracking Under Service Limit 1 STATE (S5.7.3.4)

$$f_{SA} = Z / (d_e A)^{1/3} \leq 0.6 f_y$$

$$0.6 f_y = 0.6 (60 \text{ ksi}) = 36 \text{ ksi}$$

- Calculated Values

d_e = Thickness of concrete cover measured from extreme tension fiber to center of bars located closest thereto (in).

$$\text{Top cover} = 2.5" \quad \therefore \text{We will use a cover} = 2"$$

$$d_e = 2" + \frac{\text{bar } \phi}{2}$$

$$d_e = 2 + \frac{0.625}{2} = 2.31"$$

A_c = Area of concrete having the same centroid as the principal tensile reinforcement and bounded by the surfaces of the cross-section and a straight-line parallel to the neutral axis, divided by the number of bars (in²)

$$A_c = 2 d_e (\text{spacing})$$

$$A_c = 2 (2.31") (7")$$

$$A_c = 32.34 \text{ in}^2$$

Z = crack control parameter (in/m)

$$Z = 130 \text{ in/m for severe exposure}$$

$$\therefore f_{SA} = Z / (d_e A)^{1/3}$$

$$f_{SA} = 130 / (2.31 \cdot 32.34)^{1/3}$$

$$f_{SA} = 30.87 \text{ ksi} \quad \underline{\underline{\text{Governs}}}$$

$$\therefore \text{Use } f_{SA} = \underline{\underline{30.87 \text{ ksi}}}$$

Composite Concrete Deck

14

24.) Stresses Under Service Loads (-)

Given: Modular ratio for 4 ksi concrete $\Rightarrow n = 8$ (SG. 10.3.1)

Service Live Load Moment (-)

Max unfactored Live Load (-) = -4.83 K-FT/FT (SA 4.1-1)

Load factor = 1.0 (SA 3.4.1-1)

$$M_{u,LL} = 1.0 (-4.72)$$

$$M_{u,LL} = -4.83 \text{ K-FT/FT}$$

Service Dead Load Moment (-)

• As calculated in Step #8, the Max negative unfactored dead load moments are:

$$\begin{array}{l} \text{Slab} \\ -3.116 \text{ K/FT} \end{array}$$

$$\begin{array}{l} \text{Parapet} \\ -2.146 \text{ K/FT} \end{array}$$

$$\begin{array}{l} \text{FWS} \\ -0.932 \text{ K/FT} \end{array}$$

$$M_{u,D} = 1.0(M_{u,slab}) + 1.0(M_{u,parapet}) + 1.0(M_{u,FWS})$$

$$M_{u,D} = 1.0(-3.116) + 1.0(-2.146) + 1.0(-0.932)$$

$$M_{u,D} = -6.19 \text{ K-FT/FT}$$

Total Factored Design Moment (-)

$$M_{u,TOTAL} = M_{u,D} + M_{u,LL}$$

$$M_{u,TOTAL} = -6.19 \text{ K-FT/FT} - 4.83 \text{ K-FT/FT}$$

$$M_{u,TOTAL} = -11.02 \text{ K-FT/FT}$$

• Find Transformed Moment of Inertia / d_e

Calculated Values:

$$d_e = 5.19 \text{ " (Step #21)}$$

$$A_s = \rho b d_e = 0.52 \text{ in}^2/\text{FT}$$

$$n = 8$$

$$\rho = \frac{A_s}{b d_e} = \frac{0.52}{12 \cdot 5.19} = .0083$$

Composite Concrete Deck

15

$$I_{\text{TRANSFORMED}} = \frac{1}{3} (b) (k d_c)^3 + n A_s (d_c - k d_c)^2$$

$$k = \sqrt{(p n)^2 + (2 p n)} - p n$$

$$k = \sqrt{(.0083(8))^2 + (2 \cdot .0083 \cdot 8)} - (.0083)(8)$$

$$k = 0.304$$

$$k d_c = 0.304(5.19) = 1.58''$$

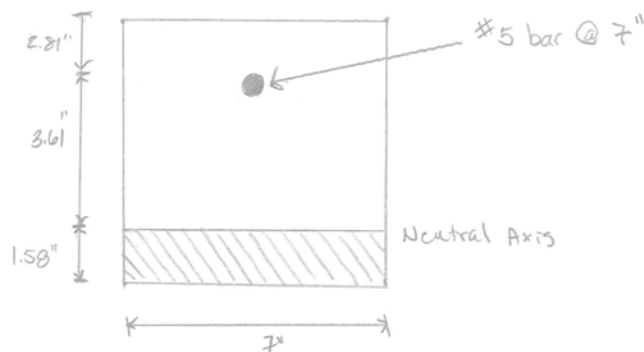
Transformed Moment of Inertia

$$I_{\text{TRANSFORMED}} = \frac{1}{3} (12) (.304 \cdot 5.19)^3 + (8)(0.52) (5.19 - .304 \cdot 5.19)^2$$

$$I_{\text{TRANSFORMED}} = 15.71 + 54.28$$

$$I_{\text{TRANSFORMED}} = 69.99 \approx 70.0 \text{ in}^4/\text{FT}$$

Figure : Schematic For Crack Control of Neg. Moment Reinforcement Under LL.



25.) Stress in Reinforcement

$$f_s = \frac{n(-M_{\text{POSAL}} \cdot b \cdot y)}{I_{\text{TRANSFORMED}}}$$

$$M_{\text{POSAL}} = -11.62 \text{ (-FT)/FT}$$

$$b = 12''$$

$$y = d_c - (k \cdot d_c) = 3.61''$$

$$n = 8$$

$$f_s = 8 \frac{(11.02 \cdot 12 \cdot 3.61)}{70}$$

$$f_s = 54.55 \text{ ksi} > f_{sa} = 30.87 \text{ ksi} \quad \phi \underline{N.6}$$

\therefore We must add Reinforcement

Try using #5 bars spaced @ 4"

$$A_c = 2 d_c (\text{spacing})$$

$$A_c = 2(2.31)(4)$$

$$A_c = 18.48 \text{ in}^2$$

$$Z = 130 \text{ K/in}$$

$$\therefore f_{sa} = Z / (d_c A)^{1/3}$$

$$f_{sa} = 130 / (2.31 \cdot 18.48)^{1/3}$$

$$f_{sa} = 37.2 \text{ ksi}$$

$$\therefore \underline{36.00 \text{ ksi}} \text{ governs}$$

We will also add reinforcement by bundling #3 bars with the #5 bars. (This will help us achieve a higher Area of Steel) We must do this because our f_s is much greater than allowed.

$$A_s' = 0.52 + 0.33 = 0.85$$

$$\rho = \frac{A_s}{b d_c} = \frac{0.85}{12 \cdot 5.19} = 0.014 \text{ in}^2/\text{ft}$$

$$k = \sqrt{(\rho n)^2 + (2 \rho n)} - \rho n$$

$$k = \sqrt{(0.014 \cdot 8)^2 + (2 \cdot 0.014 \cdot 8)} - 0.014(8)$$

$$k = 0.374$$

27) Design of Overhang

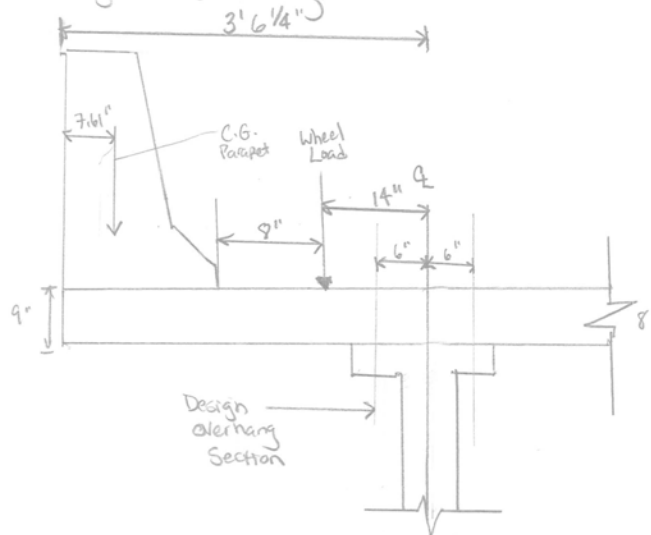


Figure : Overhang Region, Dimensions and Truck Loading

Assumed Loads:

- Self weight of Slab in overhang area = 112.5 lb/ft^2 of deck overhang S.A.
- Weight of Parapet = 650 lb/ft of length of parapet
- FWS = 30 lb/ft^2 of deck S.A.

* As required by S.A.B.4.1, there are 3 design cases which must be analyzed:

- ① Horizontal Vehicular Collision:
At inside face of Parapet

Necessary Values:

$$M_c = \text{Collision moment @ Exterior girder} = 28.21 \text{ K-FT/FT (LRFD EXAMPLE)}$$

$$\phi = 1.0 \text{ (Extreme Limit State)}$$

$$\text{Load Factor} = 1.25 \text{ (S1.3.2.1)}$$

$$M_{DL\text{slab}} = 1.25 \left(\frac{\text{Thickness} \cdot \text{density}}{2} (\text{Moment Arm})^2 \right)$$

$$M_{DL\text{slab}} = 1.25 \left(\frac{9''}{12''} \cdot 0.150 \text{ KCF} (1.6875'')^2 \right)$$

$$M_{DL\text{slab}} = 0.20 \frac{\text{K-FT}}{\text{FT}}$$

Composite Concrete Deck

19

• Parapet Moment

$$M_{DL} = 1.25 \cdot \text{Weight} (M.ARM)$$

$$M_{DL} = 1.25 \cdot 0.65 (1.053)$$

$$M_{DL} = 0.86 \text{ K-FT/FT}$$

$$\begin{aligned} \text{Moment Arm} &= 1.6875 - \frac{7.61}{12} \\ &= 1.053 \text{ ft.} \end{aligned}$$

• Total Factored Moment

$$M_{TOTAL} = M_C + M_{DLSLAB} + M_{DLPARAPET}$$

$$M_{TOTAL} = 28.21 + 0.20 + 0.86$$

$$M_{TOTAL} = 29.27 \text{ K-FT/FT}$$

• Find L_c , T , and R_w

Needed Values:

$M_b = 0 \text{ K-FT}$ For type F barriers which have no additional beam section at their top. (SA 13.2-1)

$H = \text{Height of parapet} = 3.5 \text{ FT}$

$M_w = \text{Flexural capacity about vertical axis} = 18.52 \text{ K-FT (SA 13.2-1)}$

$M_c = \text{Flexural Resistance of wall about axis II to longitudinal axis of bridge}$
 $M_c = 16.00 \frac{\text{K-FT}}{\text{FT}} \text{ (SA 13.2-1)}$

$L_t = \text{Longitudinal length of distribution of impact force, } F_t$
 $L_t = 4 \text{ ft}$

$$T = \frac{R_w}{L_c + 2H}$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_w + H)}{M_c}}$$

$$L_c = \frac{4}{2} + \sqrt{\left(\frac{4}{2}\right)^2 + \frac{8(3.5)(0 + 18.52 + 3.5)}{16.00}}$$

$$\underline{L_c = 12.84 \text{ FT}}$$

R_w = Total transverse resistance of railing :

$$R_w = \left(\frac{2}{2L_c - L_t} \right) \left(8M_b + 8M_w H + \frac{M_c L_c^2}{H} \right)$$

$$R_w = \left(\frac{2}{2(12.84 - 4)} \right) \left(8(0) + 8(18.52)(3.5) + \frac{16(12.84)^2}{3.5} \right)$$

$$R_w = 117.36 \text{ K}$$

$$\therefore \text{Use } R_w = 117.40 \text{ K}$$

• Now solve for T

$$T = \frac{R_w}{L_c + 2H} = \frac{117.40}{12.84 + 2(3.5)}$$

$$T = 5.92 \text{ K/ft}$$

28) Required Area of Reinforcing Steel

Given: Overhang thickness = 9.0" (overhang region)

#5 bar $\phi = 0.625$ "

$$d_e = \text{Thickness} - \text{Top cover} - \frac{\text{bar } \phi}{2}$$

$$d = 6.19"$$

$$b = 12"$$

$$R_n = \frac{M_{\text{TOTAL}}(b)}{(\phi_{\text{ext}} b d_e^2)}$$

$$R_n = \frac{29.27(12)}{(1)(12)(6.19)^2} = 0.76 \text{ K/in}^2$$

$$\rho = 0.85 \left(\frac{f'_c}{f_y} \right) \left[1.0 - \sqrt{1.0 - \left(\frac{2R_n}{0.85f'_c} \right)} \right]$$

$$\rho = 0.85 \left(\frac{4}{60} \right) \left[1.0 - \sqrt{1.0 - \left(\frac{2 \cdot 0.76}{0.85 \cdot 4} \right)} \right]$$

$$\rho = .0145$$

Composite Concrete Deck

21

$$A_s = \rho b d_e$$

$$A_s = 0.0145 (12)(6.19)$$

$$A_s = 1.08 \frac{\text{in}^2}{\text{ft}} \Rightarrow \text{Use } A_s = 1.24 \frac{\text{in}^2}{\text{ft}}$$

• Spacing =

$$\text{Spacing} = \frac{A_{\text{bar}}}{A_s} = \frac{0.31}{1.24} \rightarrow \#5 \text{ bars}$$

\Rightarrow Try using #5 bars @ 4"

29.) Check depth of Compression Block

$$T_a = A_s f_y$$

$$T_a = (1.24)(60)$$

$$T_a = 74.4 \text{ K/ft}$$

$$C = T_a - T$$

$$C = 74.4 - 5.92$$

$$C = 68.48 \text{ K}$$

$$a = \frac{C}{0.85 f'_c b}$$

$$a = \frac{68.48}{0.85(4)(12)}$$

$$a = 1.68 \text{ in}$$

$$M_n = T_a \left(d_e - \frac{a}{2} \right) - T \left(\frac{d_e}{2} - \frac{a}{2} \right)$$

$$M_n = 74.4 \left(6.19 - \frac{1.68}{2} \right) - 5.92 \left(\frac{6.19}{2} - \frac{1.68}{2} \right)$$

$$M_n = 32.1 \frac{\text{K} \cdot \text{ft}}{\text{ft}}$$

$$M_r = \phi_{\text{ext}} M_n$$

$$M_r = 1.0(32.1)$$

$$M_r = 32.1 \frac{\text{K} \cdot \text{ft}}{\text{ft}}$$

$$\therefore M_r \geq M_{u \text{ TOTAL}} \quad \checkmark \text{ OK}$$

$$32.1 > 29.27$$

Composite Concrete Deck

22

$$C = \frac{a}{\beta_1}$$

$$C = \frac{1.68}{0.85} \approx 1.98 \text{ m}$$

$$\frac{C}{d_e} = \frac{1.68}{6.19} = 0.27 \leq 0.42 \checkmark \text{ OK}$$

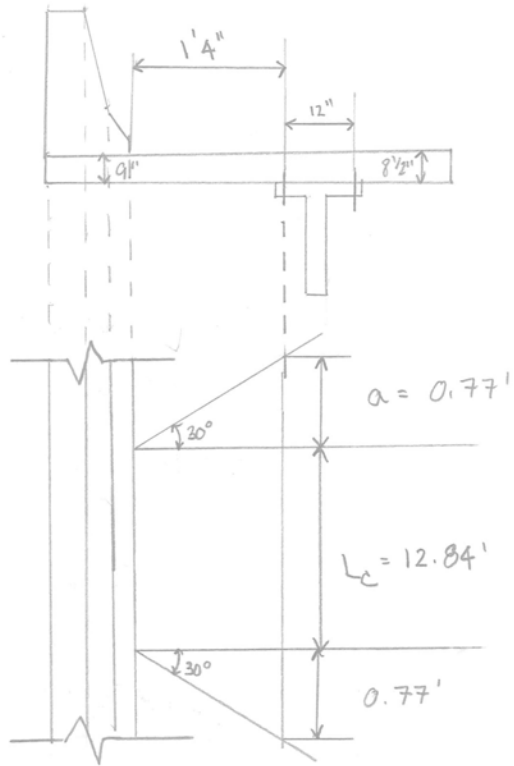
1-B Check @ Design Section in Overhang

Collision forces are distributed over a distance, L_c for moment + $L_c + 2H$ for axial force. When the design section is moved $\frac{1}{4}$ base parapet away from the girder centerline, the distribution length increases.

Assume: L_c increase based on $30^\circ \Delta$ from face of parapet.

Figure: $30^\circ \Delta$ From face of Parapet:

* The Figure is shown on the following Page



$$\tan 30^\circ = \frac{a}{16''}$$

$$16'' \tan 30^\circ = a$$

$$a = 9.24'' = 0.77'$$

For extreme limit state:

$$\phi_{ext} = 1.0$$

$$\gamma_{pDC} = 1.25$$

$$\gamma_{pDW} = 1.50$$

$$L_c = 12.84'$$

$$M_{co} = 28.21 \frac{\text{K-FT}}{\text{FT}} \left. \begin{array}{l} \\ \end{array} \right\} \text{See parapet properties}$$

$$M_{CB} = \frac{M_{co} L_c}{L_c + 2(0.77')}$$

$$M_{CB} = 25.19 \frac{\text{K-FT}}{\text{FT}}$$

Composite Concrete Deck

24

• Factored Dead Load Moment

$$M_{DC\text{-}deck} = \gamma_p DC \left[\frac{\left(\frac{9}{12}\right) (w_c) (3.021 \text{ ft})^2}{2} \right]$$

$$M_{DC\text{-}deck} = 0.64 \frac{\text{K}\cdot\text{FT}}{\text{FT}}$$

$$M_{DC\text{-}par} = \gamma_p DC \cdot w_{par} \left(3.021 - \frac{7.61}{12} \right)$$

$$M_{DC\text{-}par} = 1.94 \frac{\text{K}\cdot\text{FT}}{\text{FT}}$$

$$M_{DW\text{-}FWS} = \gamma_p DW \cdot \left[\frac{\left(\frac{2.5}{12}\right) (w_{FWS}) (3.021 - 0.77)^2}{2} \right]$$

$$w_{FWS} = .03 \frac{\text{K}}{\text{FT}} \quad M_{DW\text{-}FWS} = 0.02 \frac{\text{K}\cdot\text{FT}}{\text{FT}}$$

$$M_{\text{TOTAL}} = M_{CD} + M_{DC\text{-}deck} + M_{DC\text{-}par} + M_{DW\text{-}FWS}$$

$$M_{\text{TOTAL}} = 25.19 + 0.64 + 1.94 + 0.02$$

$$M_{\text{TOTAL}} = 27.79 \frac{\text{K}\cdot\text{FT}}{\text{FT}}$$

The Axial Tensile Force is:

$$T = \frac{R_w}{L_c + 2H_{par} + 2(0.77 \text{ FT})}$$

$$T = \frac{117.40}{12.84 + 2(3.5) + 2(0.77)}$$

$$T = 5.49 \frac{\text{K}}{\text{FT}}$$

• Overhang Slab thickness = 9.0" = t_o

Try # 5 bars:

$$\text{diameter} = 0.625 \text{ in}$$

$$d_e = t_o - \text{Top Cover} - \frac{\text{bar diameter}}{2}$$

$$d_e = 6.19 \text{ in}$$

Composite Concrete Deck

25

• The required area of reinforcing steel :

Given:

$$b = 12 \text{ in}$$

$$R_n = \frac{M_{u \text{ TOTAL}} (b)}{(\phi_{\text{ext}} b d_e^2)}$$

$$R_n = \frac{27.79 (12)}{(1.0) (12) (6.19)^2}$$

$$R_n = 0.73 \text{ ksi/in}^2$$

$$\rho = 0.85 \left(\frac{f_c}{f_y} \right) \left[1.0 - \sqrt{1.0 - \frac{(2R_n)}{0.85 f_c}} \right]$$

$$\rho = 0.85 \left(\frac{4}{60} \right) \left[1.0 - \sqrt{1.0 - \frac{(2 \cdot 0.73)}{0.85 (4)}} \right]$$

$$\rho = .0139$$

$$A_s = \rho \frac{b}{f} d_e$$

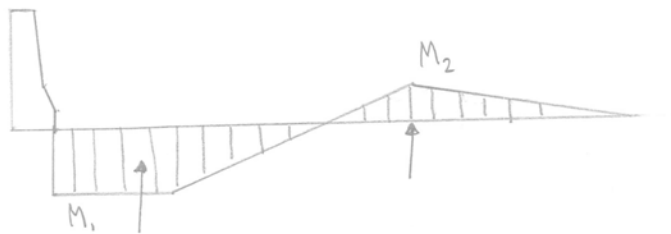
$$A_s = .0139 (12) (6.19)$$

$$A_s = 1.03 \frac{\text{in}^2}{\text{ft}}$$

The area of steel above will not be used b/c it is less than the area of steel required for that of mode of parapet

1-C Check at Design Section in First Span

Figure: Assumed Distribution of the Collision Moment Across Deck Width



Composite Concrete Deck

26

• Collision Moment at Exterior Girder

$$M_{C0} = -28.21 \frac{\text{K-FT}}{\text{FT}}$$

$$M_1 = M_{C0}$$

• Parapet Self-Weight Moment at Girder 1

$$P_{a1} = -1.17 \frac{\text{K-FT}}{\text{FT}}$$

• Parapet Self-Weight Moment at Girder 2

$$P_{a2} = 0.42 \frac{\text{K-FT}}{\text{FT}}$$

• Collision moment at $\frac{1}{4}b_f$ in Bay 1, The total collision moment:

$$M_2 = M_1 \left(\frac{P_{a2}}{P_{a1}} \right)$$

$$M_2 = -28.21 \left(\frac{0.42}{-1.17} \right)$$

$$M_2 = 10.13 \frac{\text{K-FT}}{\text{FT}}$$

By interpolation for a design section at $\frac{1}{4}b_f$ in Bay 1:

Total Collision Moment \Rightarrow

$$M_{C1/2/1} = M_{C0} + 0.25 \frac{(-M_{C0} + M_2)}{\text{spacing}}$$

$$M_{C1/2} = -28.21 + 0.25 \left(\frac{-28.21 + 10.13}{5.55} \right)$$

$$M_{C1/2} = -27.40 \frac{\text{K-FT}}{\text{FT}}$$

• Again, we will use the 30° distribution

$$\phi_{\text{ext}} = 1.0$$

$$\gamma_{\text{PDC}} = 1.25$$

$$\gamma_{\text{PDW}} = 1.50$$

$$M_{C1/2} = -27.40 \frac{\text{K-FT}}{\text{FT}}$$

$$M_{CC} = \frac{M_{C1/2} \cdot L_c}{L_c + 2(2.33)}$$

$$M_{CC} = \frac{-27.40 (12.84)}{12.84 + 2(2.33)}$$

$$M_{CC} = -26.10 \frac{\text{K-FT}}{\text{FT}}$$

• Factored dead load Moment

$$M_{DC\text{deck}} = \gamma_{pdc} (-3.116) \quad \left\{ \text{See Step \#8 for unfactored load Moments} \right\}$$

$$M_{DC\text{deck}} = -3.90 \frac{\text{K-FT}}{\text{FT}}$$

$$M_{DC\text{par}} = \gamma_{pdc} (-2.146)$$

$$M_{DC\text{par}} = -2.68 \frac{\text{K-FT}}{\text{FT}}$$

$$M_{DW\text{FWS}} = \gamma_{pdw} (-0.932)$$

$$M_{DW\text{FWS}} = -1.40 \frac{\text{K-FT}}{\text{FT}}$$

$$M_{\text{TOTAL}} = M_{DC} + M_{DC\text{deck}} + M_{DC\text{par}} + M_{DW\text{FWS}}$$

$$M_{\text{TOTAL}} = -20.1 - 3.90 - 2.68 - 1.40$$

$$M_{\text{TOTAL}} = -28.08 \frac{\text{K-FT}}{\text{FT}}$$

Axial Tensile Force:

$$T = \frac{R_w}{L_c + 2H + 2(2.33)}$$

$$T = \frac{117.40}{12.84 + (2.35) + 2(2.33)}$$

$$T = 4.79 \text{ K/FT}$$

Required Area of Reinforcing Steel

Given: Slab Thickness = 8"
5 bars $\phi = 0.625''$

$$d_c = t_s - \text{cover}_b - \frac{\text{bar } \phi}{2}$$

$$d_c = 8 - 2.5 - \frac{0.625}{2}$$

$$d_c = 5.19''$$

$$b = 12''$$

$$R_N = - \frac{M_{\text{TOTAL}} (12)}{(\phi_m b d_c^2)}$$

$$R_N = \frac{28.08 (12)}{[1 (12) (5.19)^2]}$$

$$R_N = 1.04 \text{ K/in}^2$$

$$\rho = 0.85 \left(\frac{F'_s}{F_y} \right) \left[1.0 - \sqrt{1.0 - \frac{(2R_N)}{(0.185F'_c)}} \right]$$

$$\rho = 0.85 \left(\frac{4}{60} \right) \left[1.0 - \sqrt{1.0 - \left(\frac{2 \cdot 1.04}{0.185 \cdot 4} \right)} \right]$$

$$\rho = .021$$

$$A_s = \rho b d_e$$

$$A_s = .021 (12") (5.14)$$

$$A_s = 1.31 \text{ in}^2/\text{ft}$$

Use this A_s due to the fact that it is the largest area of steel of all others listed for Case 1.

CASE 2: Design Overhang for Vertical Collision Force

• For concrete parapets, the case of vertical collision force never controls. Therefore, this procedure does not need to be considered in design.

Case 3: Design Overhang for dead and live load

3-A Check at design section in overhang

• As stated above, A_s from Case 1-C is the largest and governs.

$$\therefore A_s = 1.31 \text{ in}^2/\text{ft}$$

• As calculate in Step #15, we will use #5 bar @ 7"

$$\text{bar } \phi = 0.625"$$

$$\text{bar area} = 0.31 \text{ in}^2$$

$$A_{s\text{neg}} = \frac{\text{bar area}}{ft} \left(\frac{12}{7} \right)$$

$$A_{s\text{neg}} = 0.53 \text{ in}^2/\text{ft} < 1.31 \text{ in}^2/\text{ft} = A_s$$

• We need to add reinforcement to the overhang section by bundling one #5 and one #3 bar to each negative flexure reinforcing bar in overhang area.

$$A_s = 2(0.31 + 0.11) \left(\frac{12}{7} \right)$$

$$A_s = 1.44 \text{ in}^2/\text{ft} > 1.31 \quad \checkmark \text{ OK}$$

• Check for Cracking Under Service Limit State

In most cases, especially with deck overhang design, Cracking does not control.

• Compute Overhang Cut-off Length Requirement

$$\text{bar } \phi = 0.625 \text{ in}$$

$$\text{bar area} = 0.31 \text{ in}^2$$

$$A_s = \frac{\text{bar area}}{\text{ft}} \left(\frac{12}{7} \right)$$

$$A_s = 0.53 \frac{\text{in}^2}{\text{ft}}$$

$$d_c = t_s - \text{Cover}_t - \frac{\text{bar } \phi}{2}$$

$$d_c = 5.19 \text{ in}$$

$$T = A_s F_y$$

$$T = 0.53(60)$$

$$T = 31.8 \frac{\text{K}}{\text{ft}}$$

$$a = \frac{T}{0.85 f'_c b}$$

$$a = \frac{31.8}{0.85(4)(12)}$$

$$a = 0.78 \text{ in}$$

$$M_n = A_s f_y \left(d_c - \frac{a}{2} \right)$$

$$M_n = 0.53(60) \left(5.19 - \frac{0.78}{2} \right)$$

$$M_n = 12.72 \frac{\text{K} \cdot \text{ft}}{\text{ft}}$$

° Compute the nominal resistance for neg. Flexure

$$M_r = \phi_F \cdot M_n$$

$$M_r = 0.9 (12.72)$$

$$M_r = 11.45 \frac{\text{K-FT}}{\text{FT}}$$

Based on the nominal flexural resistance and on interpolation of the factored design moments, the theoretical cut-off point for the additional #5 bar is 3.75 feet from the centerline of the fascia girder.

$$d_e = 5.19''$$

$$15 \times \text{nominal bar } \phi = 15 (0.625) = 9.375''$$

$$1/20 \text{ of clear span} = 1/20 (5.55 \cdot 12) = 3.33''$$

$$\ast \text{ Use cut-off} = 9.5''$$

⇒ The total required length past the centerline of the fascia girder into the first bay is:

$$\text{Cut-off}_{\text{TOTAL}} = 9.5 + 3.75 (12)$$

$$\text{Cut-off}_{\text{TOTAL}} = 54.5 \text{ in}$$

° Compute Overhang Development Length

$$\text{bar } \phi = 0.625''$$

$$\text{Area bar} = 0.31 \text{ in}^2$$

$$f'_c = 4.0 \text{ ksi}$$

$$f_y = 60 \text{ ksi}$$

⇒ Development length is the larger of:

$$\frac{1.25 A_b f_y}{f'_c} = 11.63''$$

OR

$$0.4 (\text{bar } \phi) (f_y) = 15.00''$$

OR

$$12''$$

∴ Use a development length of 15.00''

◦ Modification Factors: (S6.11.2)

Epoxy Coated bars: 1.2 (S5.11.2.1.2)

Bundled Bars: 1.2 (S6.11.2.3)

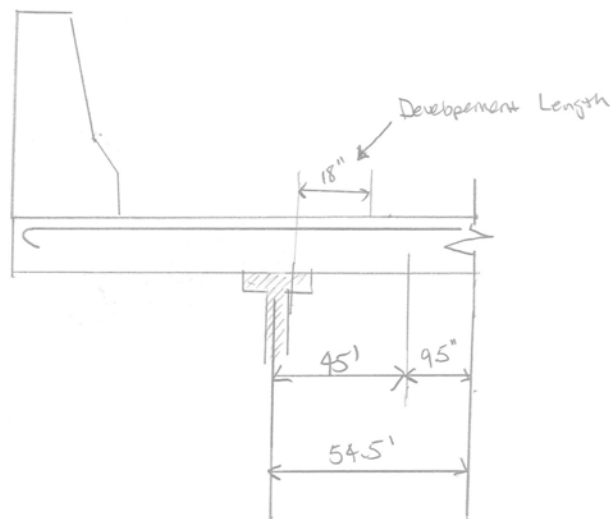
Spacing > 6 in with more than 3" of clear cover in direction of Spacing:
0.8

$$l_d = 15.00 (1.2) (1.2) (0.8)$$

$$l_d = 17.28 \text{ in.}$$

∴ Use $l_d = 18.00 \text{ in.}$

FIGURE: Length of Overhang Negative Moment Reinforcement



Composite Concrete Deck

32

• Design Bottom Longitudinal Distribution Reinforcement

$$\text{Girder Spacing} = 5.55 \text{ FT}$$

* Primary reinforcement is \perp to traffic

$$A_{s \text{ bot percent}} = \frac{220}{\sqrt{5.55}} \quad \text{where } A_{s \text{ bot long}} \leq 677.$$

$$A_{s \text{ bot percent}} = 93.47.$$

\therefore Use 677.

\Rightarrow We will use #5 bars @ 7"

$$\text{bar } \phi = 0.625"$$

$$\text{bar area} = 0.31 \text{ in}^2$$

$$A_s = \text{bar area} \left(\frac{12}{7} \right)$$

$$A_s = 0.53 \text{ in}^2$$

$$A_{s \text{ bot long}} = A_{s \text{ bot percent}} \cdot A_s$$

$$A_{s \text{ bot long}} = 0.67 (0.53)$$

$$A_{s \text{ bot long}} = 0.36 \frac{\text{in}^2}{\text{FT}}$$

• Calculate the required spacing using #5 bars:

$$\text{spacing} = \frac{\text{bar Area}}{A_{s \text{ bot long}}}$$

$$\text{spacing} = \frac{0.31}{0.36}$$

$$\text{spacing} = 0.86 \text{ FT} (12) = 10.33 \text{ in.}$$

\therefore Use spacing = 10 in.

The top longitudinal temperature and shrinkage reinforcement must satisfy:

$$A_s \geq 0.11 \frac{A_g}{f_y}$$

$$A_g = 8 \text{ in} (12)$$

$$A_g = 96 \text{ in}^2/\text{FT}$$

$$0.11 \frac{A_g}{f_y} = 0.11 \left(\frac{96}{60} \right)$$

$$= 1.76 \text{ in}^2/\text{FT}$$

Composite Concrete Deck

33

- The amount of Steel required for top longitudinal Reinforcement

$$A_{s_{req}} = \frac{.176 \frac{\text{in}^2}{\text{ft}}}{2}$$

$$A_{s_{req}} = .09 \frac{\text{in}^2}{\text{ft}}$$

- Check #4 bars @ 10" spacing

$$A_{s_{act}} = 0.2 \frac{\text{in}^2}{\text{ft}} \left(\frac{12}{10} \right)$$

$$A_{s_{act}} = 0.24 \frac{\text{in}^2}{\text{ft}}$$

$$0.24 \frac{\text{in}^2}{\text{ft}} > .09 \quad \checkmark \text{ OK}$$

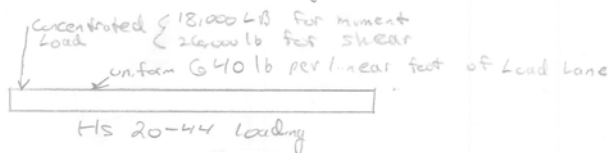
Appendix C: Composite Girder Design

Composite Girder Design

Select steel beam for composite section

determine loads

live loads - use HL 93 loading



HL 20-44 loading

HL 93 loading is HL loading & .64 k/ft lane load

total load = .64 k/ft + .64 k/ft = 1.28 k/ft load lane

use 12' wide lanes. (30' lanes on actual bridge)

1.28 k/ft lane load $\times \frac{1 \text{ lane}}{12' \text{ wide}} = .106 \text{ k/ft/ft of width}$

try 60' bays

live load is .637 k/ft

dead load = weight of deck = 60' $\left(\frac{8''}{2}\right)$ (150 lb/ft³) = 1601 k/ft

$W_U = 1.2 D_L + 1.6 L_L = 1.2(.601) + 1.6(.637) = 1.74 \text{ k/ft}$

$M_U = \frac{1.74(88)^2}{8} = 1684.32 \text{ ft} \cdot \text{k}$

$b_c = \frac{2}{3} \left(\frac{88}{12} \right) (12) = 264''$

$b_e = (2)(6.01)(12) = 144.3'' \leftarrow \text{governs}$

$Y_{con} = 8''$

assume $\alpha = 2''$

$Y_1 = 0$

$Y_2 = 8 - 2/2 = 6''$

Try W27 x 94 $A = 27.7 \text{ in}^2$ $d = 26.9''$ $t_w = .49''$

$\Sigma Q_n = 277(50) = 1385 \text{ k}$

$$\alpha = \frac{\sum Q_n}{.85 F_c b_e} = \frac{1385}{.85(4)144.3} = 2.82 < 8 \quad \text{OK}$$

$$\gamma_2 = 8 - \frac{2.82}{2} = 6.59'' \quad 2070 + \frac{.09}{.50} (2120 - 2070) = 2079 \text{ ft-k} > 1684 \text{ ft-k} \quad \therefore \text{OK}$$

Design studs

$$Q_n = \text{Table 3-21} = 17.2 \text{ k}$$

$$\sum Q_n = 1385$$

$$\text{No. of Connectors} = \frac{2 \times 1385}{17.2} = 161.04$$

= 162 studs $3/4''$ studs

Check strength of W section before concrete hardens

weight of concrete = .601 k/ft total load also add 20 psf
Construction L.L.

$$W_u = 1.6(.02 \times 6 + .601) = 1.154 \text{ k/ft}$$

$$M_u = 1.154(88^2)/8 = 1116.7 \text{ ft-k} > 1040 \text{ ft-k} \quad \text{use bigger beam}$$

try W 27x102

$$\text{when } \gamma_2 = 6.59 \quad M_{p4b} = 2260 + \frac{.09}{.5} (2310 - 2260) = 2269 \text{ ft-k} > 1684 \text{ ft-k} \quad \therefore \text{OK}$$

$$\text{Design studs} = \frac{2(1500)}{17.2} = 175 \text{ } 3/4'' \text{ studs}$$

Check before concrete hardens

$$M_u = 1116.7 \text{ ft-k} < 1140 \text{ ft-k} \quad \therefore \text{OK}$$

service load deflection before concrete hardens

$$\text{total load} = .02(6) + .601 = .721 \text{ k/ft}$$

$$M_D = \frac{.721(88)^2}{8} = 697.9 \text{ ft} \cdot \text{k}$$

$$\Delta_{DL} = \frac{5wL^4}{384EI} = \frac{5}{384ES} \left(\frac{wL^2}{8} \right) L^2 (8) = \frac{M_D L^2}{161I} = \frac{697.9(88)^2}{161(3620)} = 9.3"$$

$$9.3" > 2.5"$$

If center pier stays till after concrete hardens

$$L = 44' \quad M_D = \frac{.721(44')^2}{8} = 174.5 \text{ ft} \cdot \text{k}$$

$$\Delta_{DL} = \frac{174.5(44)^2}{161(3620)} = .58" > 2.5"$$

Service LL deflection after composite is available

$$M_L = \frac{.02(88)^2}{8} = 616.6 \text{ ft} \cdot \text{k}$$

Lower bound I from table 3-20 for W27x102

$$Y_1 = 0 \quad Y_2 = 6.59$$

$$I = 9650 + \frac{.01}{5} (9950 - 9650) = 9704.4$$

$$\Delta_{LL} = \frac{616.6(88)^2}{161(9704)} = 3.06" > \frac{L}{300} = 2.9"$$

Close, so OK

Check Shear

$$W_u = 1.2(.601) + 1.6(.637) = 1.74 \text{ k/ft}$$

$$V_u = \frac{.38(1.74)}{2} = 76.56 \text{ k} < 419 \text{ k (table 3-2)}$$

\therefore OK

Use a W 27x102 beam

exterior girder

Determine length of overhang

75.13' wide bridge 12 bay @ 6' = 72'

3.13' total overhang

$3.13/2 = 1.565'$ overhang on each side

width of effective support from exterior girder

$$3' + 1.565' = 4.565'$$

parapet is 1.6875' wide

$$4.565 - 1.6875 = 2.8775' \text{ for road traffic}$$

Dead load = wt of concrete + wt of parapet

$$= 4.565' (.150 \frac{\text{K}}{\text{ft}}) (8/12) + .650 = 1.106 \text{ K}$$

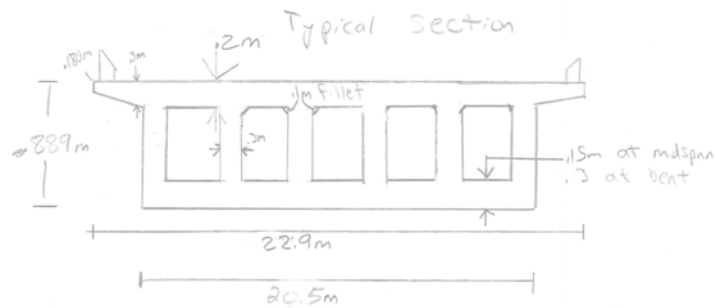
$$L_L = 2.8775 (.109) = .305 \text{ K}$$

$$w_L = 1.2 D_L + 1.6 L_L = 1.2(1.106) + 1.6(.305) = 1.8152 \text{ K/ft}$$

$$M_U = \frac{1.8152 (38)^2}{8} = 1757 \text{ ft-k} < 2269 \therefore \text{OK}$$

Appendix D: Pre-stressed Design

Prestressed concrete



Initial concrete: $f'_{ci} = 24 \text{ mpa}$, $E_{ci} = 24,768 \text{ MPa}$
 Final concrete: $f'_c = 28 \text{ mpa}$, $E_c = 26,752 \text{ MPa}$
 Prestressing steel: $f_{pu} = 1860 \text{ MPa}$ low relax strand, $E_p = 197,000 \text{ MPa}$
 Mild steel: $f_y = 400 \text{ MPa}$, $E_s = 200,000 \text{ MPa}$

Prestressing

Anchorage seat thickness = 10mm

Prestressing stress at Jacking $f_{pj} = 0.8 f_{pu} = 1488 \text{ MPa}$

Determine cross section geometry

$d = 2.889 \text{ m}$ depth

Girder spacing $S < 2d = 2(2.889) = 1.778 \text{ m}$

1.2m overhang

$22.9 - 2(1.2) = 20.5 \text{ m}$ - center to center distance of 2 exterior girders

Try 12 bays, 13 girders $20.5/12 = 1.71 \text{ m} < 1.778 \text{ m}$ ok

Section Properties

$$\text{Area: midspan} = 2.05(2) + 20.5(13) + 2.4(24) + (1 \times 1)2(12) + (2.889 - 2 - 0.15)3(12) \\ = 9.93 \text{ m}^2$$

$$\text{Bent} = 20.5(2) + 20.5(3) + 2.4(24) + (1 \times 1)2(12) + (2.889 - 2 - 0.3)3(12) \\ = 12.47 \text{ m}^2$$

Center of Gravity, y_b (m)

$$\begin{aligned} \text{mid.} \quad & \frac{20.5(2)(.789) + 2(1.2)(.24)(.78) + 3(.539)(13)(.4195) + 20.5(15)(.573)}{20.5(2) + 2(1.2)(.24) + (3)(.539)(13) + 20.5(15)} \\ & = .488 \text{ m} \end{aligned}$$

Bent

$$\begin{aligned} & \frac{20.5(2)(.789) + 2(1.2)(.24)(.78) + 3(.789)(13)(.639) + 20.5(13)(.13)}{20.5(2) + 2(1.2)(.24) + 3(.789)(13) + 20.5(13)} \\ & = .45 \text{ m} \end{aligned}$$

$$I \text{ (m}^4\text{)} = \frac{1}{2} b t^3 + b (t) y_b^2$$

$$\begin{aligned} \text{midspan} \quad & = \frac{1}{2} (20.5) (.889)^3 + 20.5 (.889) (.488 - .44)^2 - \\ & 12 \left(\frac{1}{2} (1.478) (.539)^3 + 1.478 (.539) (.488 - .4195)^2 \right) + \\ & 2 \left(\frac{1}{2} (1.2) (.24)^3 + 1.2 (.24) (.629 - .488)^2 \right) = \\ & 1.24 - .276 + .014 = .98 \text{ m}^4 \end{aligned}$$

$$\begin{aligned} \text{Bent} \quad & = \frac{1}{2} (20.5) (.889)^3 + 20.5 (.889) (.45 - .445)^2 - \\ & 12 \left(\frac{1}{2} (1.478) (.389)^3 + 1.478 (.389) (.49 - .45)^2 \right) + \\ & 2 \left(\frac{1}{2} (1.2) (.24)^3 + 1.2 (.24) (.629 - .45)^2 \right) = \\ & 1.2 - .098 + .021 = 1.12 \text{ m}^4 \end{aligned}$$

Calculate loads

a. Component dead load - DC

$$\begin{aligned} \text{DC1 - girder self weight (density } 2400 \text{ kg/m}^3\text{)} \\ \text{midspan} - 2400 \text{ kg/m } (9.81) \text{ Area}_1 &= 20.5 \text{ KN/m} \\ \text{Bent} - 2400 (9.81) (12.47) &= 25.2 \text{ KN/m} \\ \text{DC2 Parapet} - 1155 \text{ kg/m } (9.8) &= 11.325 \text{ KN/m} \\ \text{b. Wearing Surface load DW} \\ &= .075 \text{ m } (2250 \text{ kg/m}^3) (9.81) = 2.28 \text{ KN/m} \end{aligned}$$

Live-load LL & dynamic load allowance -IM:

AASHTO HL-93 vehicular LL IM = 33%

Calculate Live load distribution factors

$$d = 889 \text{ mm}$$

$$N_L = 12$$

$$S = 1710 \text{ mm}$$

$$L = 26800 \text{ mm}$$

$$W_e = 1778(5) + 1200 = 2055 \text{ mm}$$

$$D_e = 765 \text{ mm}$$

one lane loaded

$$g_m = \left(175 + \frac{S}{100}\right) \left(\frac{D_e}{L}\right)^{.35} \left(\frac{1}{N_L}\right)^{.45}$$

$$= \left(175 + \frac{1778}{100}\right) \left(\frac{765}{26800}\right)^{.35} \left(\frac{1}{12}\right)^{.45} = .223 \text{ lanes}$$

2 or more design lanes loaded

$$g_m = \left(\frac{12}{N_L}\right)^{.25} \left(\frac{S}{430}\right) \left(\frac{1}{L}\right)^{.25}$$

$$= \left(\frac{12}{12}\right)^{.25} \left(\frac{1778}{430}\right) \left(\frac{1}{26800}\right)^{.25} = .331 \text{ (controls)}$$

exterior girder

$$g_m = \frac{W_e}{1000} = \frac{2055}{4200} = .486$$

Live load distribution factor for shear

interior girder (AASHTO table 4.6.2.2.3a-1)

$$g_v = \left(\frac{S}{2900} \right)^{.6} \left(\frac{d}{L} \right)^{.1}$$

$$= \left(\frac{1701}{2900} \right)^{.6} \left(\frac{881}{26000} \right)^{.1} = .531$$

2 or more lanes loaded

$$g_v = \left(\frac{S}{2200} \right)^{.9} \left(\frac{d}{L} \right)^{.1}$$

$$= \left(\frac{1740}{2200} \right)^{.9} \left(\frac{881}{26000} \right)^{.1} = .587 \text{ - Controls}$$

exterior girder (AASHTO table 4.6.2.2.3b-1)

one design lane loaded - Lever Rule

$$R = \frac{965 + 900}{1778} = 1.05$$

$m = 1.2$ - for 1 design lane loaded

$$g_v = mR = 1.2(1.05) = 1.26 \text{ - Controls}$$

2 or more design lanes loaded

$$g_v = e g_{v, \text{interior}} = \left(0.64 + \frac{d_e}{3800} \right) g_v$$

$$= \left(0.64 + \frac{765}{3800} \right) (.587) = .49 \text{ lanes}$$

Strength limit State I

Bending moment

shear

interior girder

.336
.587

exterior girder

.4186
1.26

$$(LL+Im) = 819 + 1.3(1626.4) = 2933 \text{ kN/lane}$$

$$\text{Shear } \frac{wL}{2} = \frac{9.12(26.8)}{2} = 122 \text{ kN}$$

Used RISA



$$\sum M @ B = 0: 15.5(35.5) + (15.5 \times 4.2)14.2 + (15.5 + 4.2 \times 6.1)14.2 - 261(A)$$

$$A = 261 \text{ kN}$$

$$\text{max Shear} = 261 \text{ kN}$$

$$(LL+Im) = 122 + 1.3(261) = 461 \text{ kN}$$

Factored loads

$$\text{Strength 1} = 1.25(DL) + 1.5(DW) + 1.75(LL+Im)$$

$$\text{Service 1} = DL + DW + (LL+Im)$$

Calculate properties for interior Girder

Location	Dimension	mid.	Bent
Top Flange	hf (mm)	200	200
	Leff/4 (mm)	9000	11813
	12 hf tbw (mm)	2700	2700
	S (mm)	1700	1700
	bessf	1700	1700
Bottom	hf	150	300
	Leff	9000	11813
	12 hf tbw	2100	3900
	S	1710	1710
	bessf	170	1710
Area	A (m ²)	.775	.972
Center of gravity	Yb (m)	.467	.414
I	m ⁴	.08	.09

$$F_{cyp} = \frac{8800}{1772} + \frac{8800(1.208)^2}{.09} + \frac{(-1226)(1.208)}{.09} = 10450 \text{ kN/m}^2 = 10.4 \text{ MPa}$$

$$\Delta S_{pES} = \frac{N-1}{2N} \frac{E_p}{E_{ci}} S_{cyp} = \frac{4-1}{2(4)} \frac{197000}{24768} (0.4) = 29.5 \text{ MPa}$$

Time dependent losses table 5.9.5.3-1

$$\Delta S_{pTM} = 145 \text{ MPa}$$

Total losses

$$\Delta S_{pT} = \Delta S_{pEF} + \Delta S_{pA} + \Delta S_{pES} + \Delta S_{pTM}$$

See table

$$F_{pCI} = 1 - \frac{\Delta S_{pEF} + \Delta S_{pA} + \Delta S_{pES}}{S_{pJ}}$$

$$F_{pCF} = 1 - \frac{\Delta S_{pT}}{S_{pJ}}$$

See table

Prestress force & moment coefficients

at 13.41 m Cable path is .20m from bottom
e, is distance from cable to CG for interior girder
(negative means below)
 $e = .4167 - .20 = .2167$

$$F_{pCI} = .910 \quad F_{pCF} = .813$$

$$F_{pCI}e = .910 (-.2167)$$

$$F_{pCF}e = .813 (-.2167) = -.1762$$

Calculate Prestress losses

Frictional loss $\Delta S_{eF} = f_{pj} (1 - e^{-(Kx + \mu\alpha)})$

$K = 6.6 \times 10^{-7} / \text{mm}$

$\mu = 0.25$

x = length of prestressing tendon

α = sum of absolute values of angle changes

$= 2e_p / L_p$

e_p = vertical distance

L_p = horizontal distance

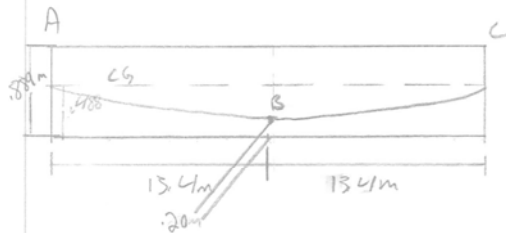
$f_{pj} = 1438 \text{ MPa}$

Anchorage set loss ΔS_{eA}

thickness $DL = 10 \text{ mm}$

$E = 200,000 \text{ MPa}$

$L_{pA} = 26.8 \text{ m}$



Parabolic cable path

$$L_{pA} = \sqrt{\frac{E A_c L_{eF}}{\Delta S_{eF}}} = \sqrt{\frac{200,000 (10) (26,800)}{22.9}} = 48.4 \text{ m} > 26.8 \text{ m}$$

$$\Delta f = \frac{2 \Delta S_{pF} L_{pA}}{L_{eF}} = \frac{2 (22.9) (48.4)}{26.8} = 82.7 \text{ MPa}$$

$$\Delta f_{pA} = \Delta f \left(1 - \frac{x}{L_{pA}}\right) = 82.7 \left(1 - \frac{x}{48.4}\right)$$

Elastic shortening loss ΔS_{pEs}

Assume $P_j = 8800 \text{ kN}$

Prestressing tendons $N = 4$

$$F_{gp} = \frac{P_j}{A} + \frac{P_j e^2}{I_x} + \frac{M D_G e}{I_x}$$

e is the distance between cable & CG. = $488.28 = 208$

determine amount of prestressing steel

See tables

Peak jacking force = 12892 kN

Use 12900 kN

$$A_{ps} = P_j / S_{ps} = 12900 / 1488 (1000) = 8670 \text{ mm}^2$$

Check Concrete Strength for interior girder - service limit state 1

$$S_{DC1} + S_{PS1} \leq 0.55 f'_c \quad \text{@ prestressing state}$$

$$S_{DC1} + S_{DC2} + S_{DW} + S_{LL} + S_{IM} + S_{PSF} \leq 0.45 f'_c \quad \text{@ service state}$$

Appendix D: Bridge Inspection

MASSACHUSETTS HIGHWAY DEPARTMENT										PAGE 1 OF 18	
2-DIST 03		B.I.N. 1LA		STRUCTURES INSPECTION FIELD REPORT				BR. DEPT. NO. W-44-063			
ROUTINE & SPECIAL MEMBER INSPECTION											
CITY/TOWN WORCESTER				8-STRUCTURE NO. W44063-1LA-MHD-NBI		11-Kilo. POINT 036.097		41-STATUS A:OPEN		90-ROUTINE INSP. DATE AUG 13, 2009	
07-FACILITY CARRIED ST122 GRAFTON ST				MEMORIAL NAME/LOCAL NAME		27-YR BUILT 1931		106-YR REBUILT 0000		YR REHAB'D (NON 106) 0000	
06-FEATURES INTERSECTED US 20 SW CUT OFF				26-FUNCTIONAL CLASS Urban Arterial		DIST. BRIDGE INSPECTION ENGINEER <i>L. A. Gauthier</i>					
43-STRUCTURE TYPE 302 : Steel Stringer/Girder				22-OWNER State Highway Agency		21-MAINTAINER State Highway Agency		TEAM LEADER <i>R. C. Angell</i>			
107-DECK TYPE 1 : Concrete Cast-in-Place				WEATHER Showers		TEMP. (air) 19°C		TEAM MEMBERS P. J. LEOVICH <i>PJL</i>			
ITEM 58			ITEM 59			ITEM 60					
DECK			SUPERSTRUCTURE			SUBSTRUCTURE					
1. Wearing surface			1. Stringers			1. Abutments					
2. Deck Condition			2. Floorbeams			a. Pedestals					
3. Stay in place forms			3. Floor System Bracing			b. Bridge Seats					
4. Curbs			4. Girders or Beams			c. Backwalls					
5. Median			5. Trusses - General			d. Breastwalls					
6. Sidewalks			a. Upper Chords			e. Wingwalls					
7. Parapets			b. Lower Chords			f. Slope Paving/Rip-Rap					
8. Railing			c. Web Members			g. Pointing					
9. Anti Missile Fence			d. Lateral Bracing			h. Footings					
10. Drainage System			e. Sway Bracings			i. Piles					
11. Lighting Standards			f. Portals			j. Scour					
12. Utilities			g. End Posts			k. Settlement					
13. Deck Joints			6. Pin & Hangers			l. Settlement					
14.			7. Conn Plt's, Gussets & Angles			m. Settlement					
15.			8. Cover Plates			2. Piers or Bents					
16.			9. Bearing Devices			a. Pedestals					
CURB REVEAL			10. Diaphragms/Cross Frames			b. Caps					
(In millimeters)			11. Rivets & Bolts			c. Columns					
E 290 W 290			12. Welds			d. Stems/Webs/Pierwalls					
			13. Member Alignment			e. Pointing					
			14. Paint/Coating			f. Footing					
			15.			g. Piles					
			Year Painted 1978			h. Scour					
			COLLISION DAMAGE: Please explain			i. Settlement					
			None () Minor () Moderate () Severe (X)			j. Settlement					
			LOAD DEFLECTION: Please explain			k. Settlement					
			None () Minor (X) Moderate () Severe ()			3. Pile Bents					
			LOAD VIBRATION: Please explain			a. Pile Caps					
			None () Minor (X) Moderate () Severe ()			b. Piles					
			Any Fracture Critical Member: (Y/N) N			c. Diagonal Bracing					
			Any Cracks: (Y/N) N			d. Horizontal Bracing					
						e. Fasteners					
						UNDERMINING (Y/N) If YES please explain N					
						COLLISION DAMAGE:					
						None (X) Minor () Moderate () Severe ()					
						SCOUR: Please explain					
						None (X) Minor () Moderate () Severe ()					
						I-60 (Dive Report): N I-60 (This Report): 5					
						93B-UW (DIVE) Insp 00/00/00					
X=UNKNOWN N=NOT APPLICABLE H=HIDDEN/INACCESSIBLE R=REMOVED											

RTN(1)7-95

CITY/TOWN WORCESTER	B.I.N. 1LA	BR. DEPT. NO. W-44-063	8-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
-------------------------------	----------------------	----------------------------------	--	--

ITEM 61 CHANNEL & CHANNEL PROTECTION <div style="text-align: right; margin-right: 10px;">N</div> <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th></th> <th>Dive</th> <th>Gur</th> <th>DEF</th> </tr> </thead> <tbody> <tr><td>1.Channel Scour</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>2.Embankment Erosion</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>3.Debris</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>4.Vegetation</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>5.Utillities</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>6.Rip-Rap/Slope Protection</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>7.Aggradation</td><td>N</td><td>N</td><td>-</td></tr> <tr><td>8.Fender System</td><td>N</td><td>N</td><td>-</td></tr> </tbody> </table> STREAM FLOW VELOCITY: Tidal () High () Moderate () Low () None (<input checked="" type="checkbox"/>)		Dive	Gur	DEF	1.Channel Scour	N	N	-	2.Embankment Erosion	N	N	-	3.Debris	N	N	-	4.Vegetation	N	N	-	5.Utillities	N	N	-	6.Rip-Rap/Slope Protection	N	N	-	7.Aggradation	N	N	-	8.Fender System	N	N	-	ITEM 36 TRAFFIC SAFETY <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th></th> <th>36</th> <th>COND</th> <th>DEF</th> </tr> </thead> <tbody> <tr><td>A. Bridge Railing</td><td>0</td><td>7</td><td>M-P</td></tr> <tr><td>B. Transitions</td><td>0</td><td>5</td><td>S-P</td></tr> <tr><td>C. Approach Guardrail</td><td>0</td><td>0</td><td>S-P</td></tr> <tr><td>D. Approach Guardrail Ends</td><td>0</td><td>0</td><td>S-P</td></tr> </tbody> </table> WEIGHT POSTING <div style="text-align: right; margin-right: 10px;"><input checked="" type="checkbox"/> Not Applicable</div> <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th></th> <th>H</th> <th>3</th> <th>3S2</th> <th>Single</th> </tr> </thead> <tbody> <tr><td>Actual Posting</td><td>N</td><td>N</td><td>N</td><td>N</td></tr> <tr><td>Recommended Posting</td><td>N</td><td>N</td><td>N</td><td>N</td></tr> </tbody> </table> Waived Date: 00/00/00 EJDMT Date: 00/00/00 Signs In Place (Y=Yes, N=No, NR=Not Required) Legibility/Visibility <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th colspan="2">At bridge</th> <th colspan="2">Other Advance</th> </tr> <tr> <th>N</th> <th>S</th> <th>N</th> <th>S</th> </tr> </thead> <tbody> <tr> <td style="text-align: center;">/</td> <td style="text-align: center;">/</td> <td style="text-align: center;">/</td> <td style="text-align: center;">/</td> </tr> </tbody> </table> CLEARANCE POSTING <div style="text-align: right; margin-right: 10px;"><input checked="" type="checkbox"/> Not Applicable</div> <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th colspan="2">E</th> <th colspan="2">W</th> <th></th> </tr> <tr> <th>ft</th> <th>in</th> <th>ft</th> <th>in</th> <th>meter</th> </tr> </thead> <tbody> <tr> <td>14</td> <td>5</td> <td>14</td> <td>8</td> <td>4.39</td> </tr> <tr> <td>14</td> <td>0</td> <td>14</td> <td>0</td> <td></td> </tr> </tbody> </table> Signs In Place (Y=Yes, N=No, NR=Not Required) Legibility/Visibility <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th colspan="2">At bridge</th> <th colspan="2">Advance</th> </tr> <tr> <th>E</th> <th>W</th> <th>E</th> <th>W</th> </tr> </thead> <tbody> <tr> <td style="text-align: center;">Y</td> <td style="text-align: center;">Y</td> <td style="text-align: center;">Y</td> <td style="text-align: center;">Y</td> </tr> <tr> <td style="text-align: center;">8</td> <td style="text-align: center;">8</td> <td style="text-align: center;">8</td> <td style="text-align: center;">7</td> </tr> </tbody> </table>		36	COND	DEF	A. Bridge Railing	0	7	M-P	B. Transitions	0	5	S-P	C. Approach Guardrail	0	0	S-P	D. Approach Guardrail Ends	0	0	S-P		H	3	3S2	Single	Actual Posting	N	N	N	N	Recommended Posting	N	N	N	N	At bridge		Other Advance		N	S	N	S	/	/	/	/	E		W			ft	in	ft	in	meter	14	5	14	8	4.39	14	0	14	0		At bridge		Advance		E	W	E	W	Y	Y	Y	Y	8	8	8	7	ACCESSIBILITY (Y/N/P) <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th></th> <th>Needed</th> <th>Used</th> </tr> </thead> <tbody> <tr><td>Lift Bucket</td><td>P</td><td>N</td></tr> <tr><td>Ladder</td><td>Y</td><td>Y</td></tr> <tr><td>Boat</td><td>N</td><td>N</td></tr> <tr><td>Waders</td><td>N</td><td>N</td></tr> <tr><td>Inspector 50</td><td>N</td><td>N</td></tr> <tr><td>Rigging</td><td>N</td><td>N</td></tr> <tr><td>Staging</td><td>N</td><td>N</td></tr> <tr><td>Traffic Control</td><td>P</td><td>N</td></tr> <tr><td>RR Flagger</td><td>N</td><td>N</td></tr> <tr><td>Police</td><td>P</td><td>N</td></tr> <tr><td>Other:</td><td>N</td><td>N</td></tr> </tbody> </table> TOTAL HOURS 8 PLANS (Y/N): Y (V.C.R.) (Y/N): N TAPE#: _____ List of field tests performed: NONE		Needed	Used	Lift Bucket	P	N	Ladder	Y	Y	Boat	N	N	Waders	N	N	Inspector 50	N	N	Rigging	N	N	Staging	N	N	Traffic Control	P	N	RR Flagger	N	N	Police	P	N	Other:	N	N
	Dive	Gur	DEF																																																																																																																																																										
1.Channel Scour	N	N	-																																																																																																																																																										
2.Embankment Erosion	N	N	-																																																																																																																																																										
3.Debris	N	N	-																																																																																																																																																										
4.Vegetation	N	N	-																																																																																																																																																										
5.Utillities	N	N	-																																																																																																																																																										
6.Rip-Rap/Slope Protection	N	N	-																																																																																																																																																										
7.Aggradation	N	N	-																																																																																																																																																										
8.Fender System	N	N	-																																																																																																																																																										
	36	COND	DEF																																																																																																																																																										
A. Bridge Railing	0	7	M-P																																																																																																																																																										
B. Transitions	0	5	S-P																																																																																																																																																										
C. Approach Guardrail	0	0	S-P																																																																																																																																																										
D. Approach Guardrail Ends	0	0	S-P																																																																																																																																																										
	H	3	3S2	Single																																																																																																																																																									
Actual Posting	N	N	N	N																																																																																																																																																									
Recommended Posting	N	N	N	N																																																																																																																																																									
At bridge		Other Advance																																																																																																																																																											
N	S	N	S																																																																																																																																																										
/	/	/	/																																																																																																																																																										
E		W																																																																																																																																																											
ft	in	ft	in	meter																																																																																																																																																									
14	5	14	8	4.39																																																																																																																																																									
14	0	14	0																																																																																																																																																										
At bridge		Advance																																																																																																																																																											
E	W	E	W																																																																																																																																																										
Y	Y	Y	Y																																																																																																																																																										
8	8	8	7																																																																																																																																																										
	Needed	Used																																																																																																																																																											
Lift Bucket	P	N																																																																																																																																																											
Ladder	Y	Y																																																																																																																																																											
Boat	N	N																																																																																																																																																											
Waders	N	N																																																																																																																																																											
Inspector 50	N	N																																																																																																																																																											
Rigging	N	N																																																																																																																																																											
Staging	N	N																																																																																																																																																											
Traffic Control	P	N																																																																																																																																																											
RR Flagger	N	N																																																																																																																																																											
Police	P	N																																																																																																																																																											
Other:	N	N																																																																																																																																																											

RATING Rating Report (Y/N): Y Date: 10/01/1984 Inspection data at time of existing rating 158: 8 159: 7 160: 7 Date: 02/04/1982	(To be filled out by DBIE) Request for Rating or Rerating (Y/N): N REASON: _____ <div style="text-align: right; margin-right: 10px;">If YES please give priority:</div> HIGH () MEDIUM () LOW ()
---	---

CONDITION RATING GUIDE (For Items 58, 59, 60 and 61)		
CODE	CONDITION	DEFECTS
N	NOT APPLICABLE	
G 9	EXCELLENT	Excellent condition.
G 8	VERY GOOD	No problem noted.
G 7	GOOD	Some minor problems.
F 6	SATISFACTORY	Structural elements show some minor deterioration.
F 5	FAIR	All primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
P 4	POOR	Advance section loss, deterioration, spalling or scour.
P 3	SERIOUS	Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
C 2	CRITICAL	Advance deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
C 1	"IMMINENT" FAILURE	Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put it back in light service.
0	FAILED	Out of service - beyond corrective action.

DEFICIENCY REPORTING GUIDE	
DEFICIENCY: A defect in a structure that requires corrective action.	
CATEGORIES OF DEFICIENCIES:	
M= Minor Deficiency: Deficiencies which are minor in nature, generally do not impact the structural integrity of the bridge and could easily be repaired. Examples include but are not limited to: Spalled concrete, Minor pot holes, Minor corrosion of steel, Minor scouring, Clogged drainage, etc.	
S= Severe/Major Deficiency: Deficiencies which are more extensive in nature and need more planning and effort to repair. Examples include but are not limited to: Moderate to major deterioration in concrete, Exposed and corroded rebars, Considerable settlement, Considerable scouring or undermining, Moderate to extensive corrosion to structural steel with measurable loss of section, etc.	
C-S= Critical Structural Deficiency: A deficiency in a structural element of a bridge that poses an extreme unsafe condition due to the failure or imminent failure of the element which will affect the structural integrity of the bridge.	
C-H= Critical Hazard Deficiency: A deficiency in a component or element of a bridge that poses an extreme hazard or unsafe condition to the public, but does not impair the structural integrity of the bridge. Examples include but are not limited to: Loose concrete hanging down over traffic or pedestrians, A hole in a sidewalk that may cause injuries to pedestrians, Missing section of bridge railing, etc.	
URGENCY OF REPAIR:	
I = Immediate-	(Inspector(s) immediately contact District Bridge Inspection Engineer (DBIE) to report the Deficiency and to receive further instruction from him/her).
A = ASAP-	(Action/Repair should be initiated by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) upon receipt of the Inspection Report).
P = Prioritize-	(Should be prioritized by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) and repairs made when funds and/or manpower is available).

RTB(2)04-07

MASSACHUSETTS HIGHWAY DEPARTMENT

PAGE 3 OF 18

STRUCTURES INSPECTION FIELD REPORT

2-DIST 03	B.I.N. 1LA	ROUTINE & SPECIAL MEMBER INSPECTION				BR. DEPT. NO. W-44-063															
CITY/TOWN WORCESTER		8-STRUCTURE NO. W44063-1LA-MHD-NBI		11-Kilo. POINT 036.097		90-ROUTINE INSP. DATE Aug 13, 2009		93*-SPEC. MEMB. INSP. DATE Aug 13, 2009													
07-FACILITY CARRIED ST122 GRAFTON ST		MEMORIAL NAME/LOCAL NAME		27-YR BUILT 1931		106-YR REBUILT 0000		*YR REHAB'D (NON 106) 0000													
06-FEATURES INTERSECTED US 20 SW CUT OFF		26-FUNCTIONAL CLASS Urban Arterial		DIST. BRIDGE INSPECTION ENGINEER <i>L. A. Gauthier</i>																	
43-STRUCTURE TYPE 302 : Steel Stringer/Girder		22-OWNER State Highway Agency		21-MAINTAINER State Highway Agency		TEAM LEADER R. C. Angelo															
107-DECK TYPE 1 : Concrete Cast-in-Place		WEATHER Showers		TEMP. (air) 19°C		TEAM MEMBERS P. J. LEOVICH															
WEIGHT POSTING		Not Applicable <input checked="" type="checkbox"/>		At bridge		Advance		PLANS (Y/N): Y													
Actual Posting		<table border="1"> <tr> <td>H</td> <td>3</td> <td>3S2</td> <td>Single</td> </tr> <tr> <td>N</td> <td>N</td> <td>N</td> <td>N</td> </tr> </table>		H	3	3S2	Single	N	N	N	N	Signs In Place (Y=Yes, N=No, NR=Not Required)		<table border="1"> <tr> <td>N</td> <td>S</td> </tr> <tr> <td></td> <td></td> </tr> </table>		N	S			(V.C.R.) (Y/N): N	
H	3	3S2	Single																		
N	N	N	N																		
N	S																				
Recommended Posting		<table border="1"> <tr> <td>N</td> <td>N</td> <td>N</td> <td>N</td> </tr> </table>		N	N	N	N	Legibility/Visibility		<table border="1"> <tr> <td></td> <td></td> </tr> <tr> <td></td> <td></td> </tr> </table>						TAPE#:					
N	N	N	N																		
Waived Date: 00/00/00		EJDMT Date: 00/00/00																			
RATING		Rating Report (Y/N): Y		Date: 10/01/1984		Request for Rating or Rerating (Y/N): N		If YES please give priority: HIGH () MEDIUM () LOW ()													
Inspection data at time of existing rating 158: 8 159: 7 160: 7 162: -		Date: 02/04/1982		REASON: <i>2</i>																	
SPECIAL MEMBER(S):																					
	MEMBER	CRACK (Y/N)	WELD'S CONDITION (0-9)	LOCATION OF CORROSION, SECTION LOSS (%), CRACKS, COLLISION DAMAGE, STRESS CONCENTRATION, ETC.	CONDITION PREVIOUS (0-9)	CONDITION PRESENT (0-9)	INV. RATING OF MEMBER			Deficiencies											
	A Item 59.4 - Girders or Beams	N	N	See remarks in comments section.	3	3	28	34	53	S-P											
	B																				
	C																				
	D																				
	E																				
List of field tests performed: NONE					<table border="1"> <tr> <td>I-58</td> <td>I-59</td> <td>I-60</td> <td>I-62</td> </tr> <tr> <td>7</td> <td>3</td> <td>5</td> <td>-</td> </tr> <tr> <td>7</td> <td>3</td> <td>5</td> <td>-</td> </tr> </table>					I-58	I-59	I-60	I-62	7	3	5	-	7	3	5	-
I-58	I-59	I-60	I-62																		
7	3	5	-																		
7	3	5	-																		
					(Overall Previous Condition)																
					(Overall Current Condition)																
DEFICIENCY: A defect in a structure that requires corrective action.																					
CATEGORIES OF DEFICIENCIES:																					
M= Minor Deficiency- Deficiencies which are minor in nature, generally do not impact the structural integrity of the bridge and could easily be repaired. Examples include but are not limited to: Spalled concrete, Minor pot holes, Minor corrosion of steel, Minor scouring, Clogged drainage, etc.																					
S= Severe/Major Deficiency- Deficiencies which are more extensive in nature and need more planning and effort to repair. Examples include but are not limited to: Moderate to major deterioration in concrete, Exposed and corroded rebars, Considerable settlement, Considerable scouring or undermining, Moderate to extensive corrosion to structural steel with measurable loss of section, etc.																					
C-S= Critical Structural Deficiency - A deficiency in a structural element of a bridge that poses an extreme unsafe condition due to the failure or imminent failure of the element which will affect the structural integrity of the bridge.																					
C-H= Critical Hazard Deficiency - A deficiency in a component or element of a bridge that poses an extreme hazard or unsafe condition to the public, but does not impair the structural integrity of the bridge. Examples include but are not limited to: Loose concrete hanging down over traffic or pedestrians, A hole in a sidewalk that may cause injuries to pedestrians, Missing section of bridge railing, etc.																					
URGENCY OF REPAIR:																					
I = Immediate- Inspector(s) immediately contact District Bridge Inspection Engineer (DBIE) to report the Deficiency and to receive further instruction from him/her.																					
A = ASAP- Action/Repair should be initiated by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) upon receipt of the Inspection Report.																					
P = Prioritize- Should be prioritized by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) and repairs made when funds and/or manpower is available.																					
X=UNKNOWN N=NOT APPLICABLE H=HIDDEN/INACCESSIBLE R=REMOVED																					

F.C.(1)7-98

CITY/TOWN WORCESTER	B.I.N. 1LA	BR. DEPT. NO. W-44-063	8-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
-------------------------------	----------------------	----------------------------------	--	--

REMARKS

BRIDGE ORIENTATION
The approaches are South to North and the elevations are West to East. This is a two span steel beam bridge with thirteen beams and twelve bays in each span numbered from West to East. The spans are numbered from South to North. There is a solid concrete pierwall.

GENERAL REMARKS
Note:
There is heavy vegetation growing at all four corners of bridge. This vegetation is starting to encroach on the superstructure of the bridge.

ITEM 58 - DECK

Item 58.1 - Wearing surface
The center paving joint is starting to open slightly. The bit. concrete wearing surface shows minor to moderate transverse and longitudinal cracking through out, heaviest along the centerline of the roadway.

Item 58.2 - Deck Condition
The underside of concrete deck shows isolated areas of minor honeycombing and transverse hairline cracking, some with efflorescence.

Item 58.6 - Sidewalks
Both concrete sidewalks show several transverse hairline cracks from almost full width to full width. Several of these cracks extend through to the underside of the sidewalks. Both sidewalks show several scale areas through out, heaviest along the granite curbs.

Item 58.7 - Parapets
There is an up to 3 in. deep section of concrete chipped off along the entire length on the bottom of the West parapet. See photo 1.

Item 58.8 - Railing
There are several nuts missing on the East AL-3 bridge rail posts, at base, mostly at the North end.

Item 58.9 - Anti Missile Fence
There is heavy vegetation growing through the anti missile fencing at the Northeast and Southwest corners. See photo #1.

Item 58.13 - Deck Joints
The concrete deck joint headers show moderate spalling, scaling and pop outs to all three joints. There is a 1 ft. section of steel missing to the South abutment joint in the Southbound travel lane. See photo #2. There are two 1 ft. sections of steel missing at the pier joint, just West of the centerline. See photo #3. There is a 4 ft. section of steel missing at the North deck joint just West of the centerline, resulting in a jagged edge. See photo #4.

APPROACHES

Approaches a - Appr. pavement condition
Both bit. conc. approach pavement shows minor longitudinal and moderate transverse cracking in several areas through out. Some of the transverse cracking is full width.

REM. (2)7-09

CITY/TOWN WORCESTER	B.I.N. 1LA	BR. DEPT. NO. W-44-063	8-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
-------------------------------	----------------------	----------------------------------	--	--

REMARKS

BRIDGE ORIENTATION
The approaches are South to North and the elevations are West to East. This is a two span steel beam bridge with thirteen beams and twelve bays in each span numbered from West to East. The spans are numbered from South to North. There is a solid concrete pierwall.

GENERAL REMARKS
Note:
There is heavy vegetation growing at all four corners of bridge. This vegetation is starting to encroach on the superstructure of the bridge.

ITEM 58 - DECK

Item 58.1 - Wearing surface
The center paving joint is starting to open slightly. The bit. concrete wearing surface shows minor to moderate transverse and longitudinal cracking through out, heaviest along the centerline of the roadway.

Item 58.2 - Deck Condition
The underside of concrete deck shows isolated areas of minor honeycombing and transverse hairline cracking, some with efflorescence.

Item 58.6 - Sidewalks
Both concrete sidewalks show several transverse hairline cracks from almost full width to full width. Several of these cracks extend through to the underside of the sidewalks. Both sidewalks show several scale areas through out, heaviest along the granite curbs.

Item 58.7 - Parapets
There is an up to 3 in. deep section of concrete chipped off along the entire length on the bottom of the West parapet. See photo 1.

Item 58.8 - Railing
There are several nuts missing on the East AL-3 bridge rail posts, at base, mostly at the North end.

Item 58.9 - Anti Missile Fence
There is heavy vegetation growing through the anti missile fencing at the Northeast and Southwest corners. See photo #1.

Item 58.13 - Deck Joints
The concrete deck joint headers show moderate spalling, scaling and pop outs to all three joints. There is a 1 ft. section of steel missing to the South abutment joint in the Southbound travel lane. See photo #2. There are two 1 ft. sections of steel missing at the pier joint, just West of the centerline. See photo #3. There is a 4 ft. section of steel missing at the North deck joint just West of the centerline, resulting in a jagged edge. See photo #4.

APPROACHES

Approaches a - Appr. pavement condition
Both bit. conc. approach pavement shows minor longitudinal and moderate transverse cracking in several areas through out. Some of the transverse cracking is full width.

REM. (2)7-09

CITY/TOWN WORCESTER	B.I.N. 1LA	BR. DEPT. NO. W-44-063	8-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
-------------------------------	----------------------	----------------------------------	--	--

REMARKS

ITEM 59 - SUPERSTRUCTURE

Item 59.4 - Girders or Beams
 Beams #1 through #10, in span #1, have collision damage that varies from minor scrapes to moderate twisting and gouging of the steel. The diaphragms, nuts, bolts, and welds have minor to severe damage in several of the collision areas. All collision areas in span #1 are unchanged since the last Special Member Inspection dated 8/14/2008. See Sketches #1 and #2, and Photos #5 through #8. Beams #1 through #4 in span #2 have minor collision scrapes on the underside of the bottom flanges over the first travel lane. The beams in both spans show moderate to heavy surface rusting in many areas throughout. The heaviest surface rusting is in the collision areas of span #1, where there is minor rust flaking of steel.

Item 59.7 - Conn Pl't's, Gussets & Angles
 Re: Item #59.1.

Item 59.9 - Bearing Devices
 Most concrete bearing encasements on the South abutment show moderate cracking and are starting to break off in several areas.

Item 59.10 - Diaphragms/Cross Frames
 Re: Item #59.1.

Item 59.11 - Rivets & Bolts
 Re: Item #59.1.

Item 59.12 - Welds
 Re: Item #59.1.

Item 59.13 - Member Alignment
 Re: Item #59.1.

Item 59.14 - Paint/Coating
 All superstructure paint is exposing many areas of moderate to heavy surface rusting, heaviest in the collision areas to span #1.

SuperStructure Collision Notes
 There is severe collision damage to many stringers in span #1. Re: Item #59.4 for collision remarks.

SuperStructure Load Deflection Notes
 Minor under heavy live loads.

SuperStructure Load Vibration Notes
 Minor under heavy live loads.

REM (2)7-99

CITY/TOWN WORCESTER	B.T.N. 1LA	BR. DEPT. NO. W-44-063	8-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
-------------------------------	----------------------	----------------------------------	--	--

REMARKS

ITEM 60 - SUBSTRUCTURE

Item 60.1 - Abutments

Item 60.1.b - Bridge Seats
 There is moderate to heavy spall/scaling to the West 1/3 of the South abutment seat and under bay #11. See photos #9 & #10. There is heavy spall/scaling to the North abutment bridge seat, under beams #2 and #9. See photos #11 & #12.

Item 60.1.c - Backwalls
 There is a severe spall/scale area, 5 ft. long x full height x 2 in. deep, to the West end of the South backwall. See photo #13. The North end of the West backwall has a scale area 6 in. long x full height x 2 in. deep.

Item 60.1.d - Breastwalls
 The South breastwall shows several areas of severe spall/scaling at both ends, heaviest at the West half. See photo #14. There is moderate map cracking at the East end of the South breastwall. There is minor full height map cracking to the East end of the North breastwall. Both breastwalls show several areas of minor to moderate honey combing throughout.

Item 60.1.e - Wingwalls
 There is moderate to heavy vegetation growth partially obscuring all four wings. The Northeast wing shows several areas of heavy spall/scaling. See photo #15. The remaining wings show minor horizontal and vertical hairline cracking in several areas through out. The Southwest wing shows minor scaling at the top, interface of the backwall.

Item 60.2 - Piers or Bents

Item 60.2.b - Caps
 There is moderate to heavy spall/scaling to the top and bottom of the West concrete pier cap nose. See photo #16. There is a 3 ft. long x 8 in. high x 3 in. deep spall/scale area at the top North face, at the East end. See photo #17.

Item 60.2.d - Stems/Webs/Pierwalls
 Both faces of pierwall show minor to heavy scaling and honeycombing. Both pierwall noses show minor to heavy scaling and honeycombing.

TRAFFIC SAFETY

Item 36a - Bridge Railing
 Both bridgerails consist of AL-3 rails with aluminum posts. Re; item #58.8 for condition remarks.

Item 36b - Transitions
 There is a solid concrete wall at all 4 corners, butted up, but not connected to AL-3 bridgerail, serving as transitions. The North end of the solid concrete wall (transition) shows a full height x 1 ft. wide area of collision damage at the Northeast corner of bridge. See photo #18. The Southwest and Northwest corners have no approach or terminal rails, (They abut the highway on/off ramps). The Southeast corner has concrete posts and 2 in. x 6 in. timber rails. Several of the posts are heavily spalled and broken and some timber rails are cracked, rotted and broken. The Northeast corner has a 6 ft. high masonry wall (private) extending to the abutter's driveway.

REM (2)7-98

CITY/TOWN WORCESTER	B.I.N. 1LA	BR. DEPT. NO. W-44-063	8-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
REMARKS				
<p><u>Item 36c - Approach Guardrail</u> Re: Item #36.b.</p> <p><u>Item 36d - Approach Guardrail Ends</u> Re: Item #36.b.</p> <p><u>Sketch / Photo Log</u> Sketch 1 : Collision damage to beams in span #1. Sketch 2 : Collision damage to beams in span #1. Photo 1 : Vegetation growing through the anti missile fence, typical at the Southwest & Northeast corners of bridge. Photo 2 : Spalling to concrete headers and missing armored steel to the South abutment deck joint. Photo 3 : Spalling to concrete headers and missing armored steel to the pier deck joint. Photo 4 : Spalling to concrete headers and missing armored steel to the North abutment deck joint. Photo 5 : Collision damage to beam #1 (West fascia), span #1. Photo 6 : Collision damage to beam #1 & beam #2, span #1. Photo 7 : Collision damage to beam #5, span #1. Photo 8 : Collision damage to beam #10, span #1. Photo 9 : Spall/scale to the West 1/3 of the South abutment seat. Photo 10 : Spall/scale to the South abutment seat under bay #11. Photo 11 : Spall/scale to the North abutment seat under bay #2. Photo 12 : Spall/scale to the North abutment seat under bay #9. Photo 13 : Spall/scale to the West end of the South backwall.. Photo 14 : Spall/scale to the West half of the South breastwall. Photo 15 : Spall/scale to the Northeast wingwall. Photo 16 : Spall/scale to the West nose of pier cap.. Photo 17 : Spall/scale to the top North face of pier cap, at the East end. Photo 18 : Collision damage at the North end of the Northeast transition wall.</p>				

REM (2)7-69

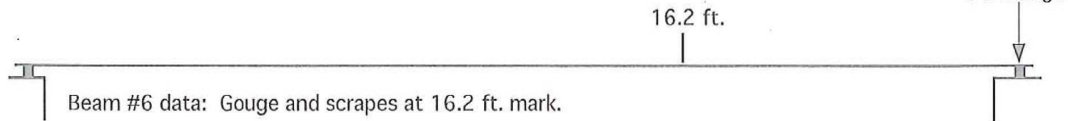
CITY/TOWN	B.I.N.	BR. DEPT. NO.	8-STRUCTURE NO.	INSPECTION DATE
WORCESTER	1LA	W-44-063	W44063-1LA-MHD-NBI	AUG 13, 2009

SKETCHES

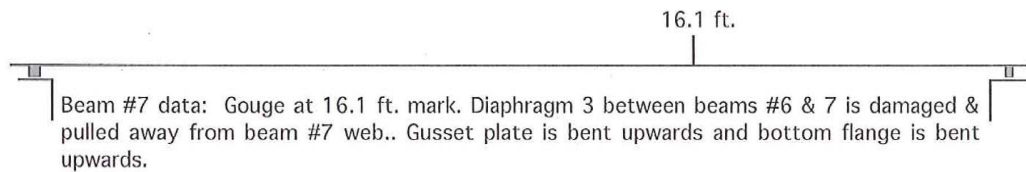
Beam #6 From West

Not To Scale

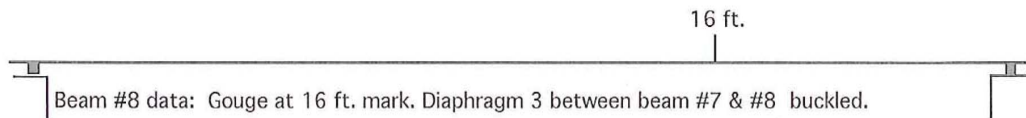
Note: All measurements taken from center-line of south bearing.



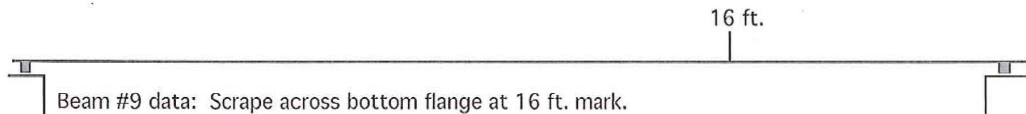
Beam #7 From West



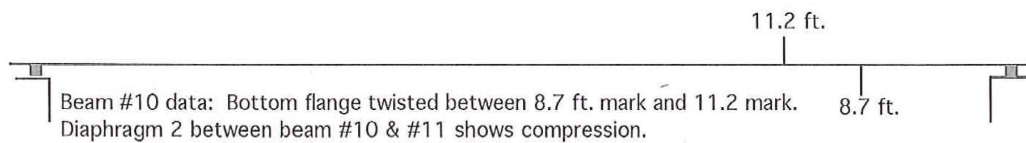
Beam #8 From West



Beam #9 From West



Beam #10 From West



No visible damage to remaining beams.

Sketch 2: Collision damage to beams in span #1.

CITY/TOWN WORCESTER	B.I.N. 1LA	BR. DEPT. NO. W-44-063	S-STRUCTURE NO. W44063-1LA-MHD-NBI	INSPECTION DATE AUG 13, 2009
------------------------	---------------	---------------------------	---------------------------------------	---------------------------------

PHOTOS



Photo 1: Vegetation growing through the anti missile fence, typical at the Southwest & Northeast corners of bridge.



Photo 2: Spalling to concrete headers and missing armored steel to the South abutment deck joint.

REM (2)7-59

The remaining pictures were omitted to reduce the size

Appendix E: Life-Cycle Cost Break Down

year	0	frequency of repairs	1	2	3	4	5	6	7	8
Non - Destructive Asphalt Testing	1320	4	1359.6	1400.388	1442.4	1485.672	1530.242	1576.149	1623.434	1672.137
Repave/milling	13200	15	13596	14003.88	14424	14856.72	15302.42	15761.49	16234.34	16721.37
Wearing Surface Sealing	1320	4	1359.6	1400.388	1442.4	1485.672	1530.242	1576.149	1623.434	1672.137
Bridge Inspection	800	2	824	848.72	874.1816	900.407	927.4193	955.2418	983.8991	1013.416
Steel Painting	68000	15	70040	72141.2	74305.44	76534.6	78830.64	81195.56	83631.42	86140.37
Concrete Patch and Replar	33000	15	33990	35009.7	36059.99	37141.79	38256.04	39403.73	40585.84	41803.41
	year	0	1	2	3	4	5	6	7	8
	cost									
15 year interval	Money	185000	196100	207866	220338	233558.2	247571.7	262426	278171.6	294861.9
	Cost			848.72		900.407		955.2418		1013.416
2 Year interval	Money	12650	13409	13313.9	14112.73	14005.06	14845.37	14723.53	15606.94	15469.14
	cost					2971.343				3344.273
4 year interval	Money	20100	21306	22584.36	23939.42	22226.16	23559.73	24973.32	26471.72	24515.09
sum of initial money		217750								
return rate										
	1.06									

	9	10	11	12	13	14	15	16	17	18	19	20	21	22
1722.301	1773.97	1827.189	1882.004	1938.465	1996.618	2056.517	2118.212	2181.759	2247.212	2314.628	2384.067	2455.589	2529.256	
17223.01	17739.7	18271.89	18820.04	19384.65	19966.18	20565.17	21182.12	21817.59	22472.12	23146.28	23840.67	24555.89	25292.56	
1722.301	1773.97	1827.189	1882.004	1938.465	1996.618	2056.517	2118.212	2181.759	2247.212	2314.628	2384.067	2455.589	2529.256	
1043.819	1075.133	1107.387	1140.609	1174.827	1210.072	1246.374	1283.765	1322.278	1361.946	1402.805	1444.889	1488.236	1532.883	
88724.58	91386.31	94127.9	96951.74	99860.29	102856.1	105941.8	109120	112393.6	115765.4	119238.4	122815.6	126500	130295	
43057.52	44349.24	45679.72	47050.11	48461.61	49915.46	51412.92	52955.31	54543.97	56180.29	57865.7	59601.67	61389.72	63231.41	
9	10	11	12	13	14	15	16	17	18	19	20	21	22	
						177919.9								
312553.6	331306.8	351185.2	372256.3	394591.7	418267.2	254768.2	270054.3	286257.5	303433	321639	340937.3	361393.6	383077.2	
	1075.133		1140.609		1210.072		1283.765		1361.946		1444.889		1532.883	
16397.29	16241.48	17215.97	17039.89	18062.28	17863.34	18935.14	18710.46	19833.09	19579.41	20754.17	20467.84	21695.91	21372.81	
			3764.009				4236.425				4768.134			
25985.99	27545.15	29197.86	26959.89	28577.48	30292.13	32109.66	29545.62	31318.36	33197.46	35189.31	32246.45	34181.24	36232.11	

	23	24	25	26	27	28	29	30	31	32	33	34	35
	2605.134	2683.288	2763.787	2846.7	2932.101	3020.065	3110.666	3203.986	3300.106	3399.109	3501.083	3606.115	3714.298
	26051.34	26832.88	27637.87	28467	29321.01	30200.65	31106.66	32039.86	33001.06	33991.09	35010.83	36061.15	37142.98
	2605.134	2683.288	2763.787	2846.7	2932.101	3020.065	3110.666	3203.986	3300.106	3399.109	3501.083	3606.115	3714.298
	1578.869	1626.235	1675.022	1725.273	1777.031	1830.342	1885.252	1941.81	2000.064	2060.066	2121.868	2185.524	2251.09
	134203.9	138230	142376.9	146648.2	151047.7	155579.1	160246.5	165053.8	170005.5	175105.6	180358.8	185769.6	191342.6
	65128.35	67082.21	69094.67	71167.51	73302.54	75501.61	77766.66	80099.66	82502.65	84977.73	87527.06	90152.87	92857.46
	23	24	25	26	27	28	29	30	31	32	33	34	35
	406061.8	430425.5	456251	483626.1	512643.7	543402.3	576006.4	316741.8	335746.3	355891.1	377244.6	399879.3	423872
								277193.4					
		1626.235		1725.273		1830.342		1941.81		2060.066		2185.524	
	22655.18	22290.68	23628.12	23217.02	24610.04	24146.48	25595.27	25072.67	26577.03	25987.98	27547.26	26883.44	28496.44
		5366.576				6040.129				6798.218			
	38406.04	35021.83	37123.14	39350.53	41711.56	37811.71	40080.42	42485.24	45034.36	40530.31	42962.12	45539.85	48272.24

	50	51	52	53	54	55	56	57	58	59	60	61	62
5786.756	5960.359	6139.169	6323.344	6513.045	6708.436	6909.689	7116.98	7330.489	7550.404	7776.916	8010.224	8250.53	
57867.56	59603.59	61391.69	63233.44	65130.45	67084.36	69096.89	71169.8	73304.89	75504.04	77769.16	80102.24	82505.3	
5786.756	5960.359	6139.169	6323.344	6513.045	6708.436	6909.689	7116.98	7330.489	7550.404	7776.916	8010.224	8250.53	
3507.125	3612.339	3720.709	3832.33	3947.3	4065.719	4187.69	4313.321	4442.721	4576.002	4713.282	4854.681	5000.321	
298105.6	307048.8	316260.2	325748	335520.5	345586.1	355953.7	366632.3	377631.3	388960.2	400629	412647.9	425027.3	
144668.9	149009	153479.2	158083.6	162826.1	167710.9	172742.2	177924.5	183262.2	188760.1	194422.9	200255.6	206263.3	
50	51	52	53	54	55	56	57	58	59	60	61	62	
										672821.1			
403234.8	427428.9	453074.6	480259.1	509074.6	539619.1	571996.2	606316	642695	681256.7	8941.722			
3507.125		3720.709		3947.3		4187.69		4442.721		4713.282		5000.321	
31617.58	33514.63	31581.56	33476.45	31300.9	33178.96	30730.74	32574.59	29819.78	31608.96	28509.42	30219.99	26732.85	
		12278.34				13819.38				15553.83			
53435.25	56641.36	47024.81	49846.29	52837.07	56007.3	44719.19	47402.34	50246.49	53261.27	39969.89	42368.08	44910.17	

